ACI News Letter

Vol. 23, No. 9 JOURNAL of the AMERICAN CONCRETE INSTITUTE May 1952

Double arch culvert on a western railroad after rehabilitation by Intrusion-Prepakt. The high bonding strength and density of Prepakt Concrete provide the renewed structure with maximum resistance to all forms of deterioration.

Cracks and extreme surface deterioration of the old structure made it necessary to chip and clean away deteriorated materials down to sound concrete. Repair operations were carried out with no interruption to rail traffic.

PREPAKT—

the method for lower railroad maintenance costs...

Here is another case where PREPAKT methods and materials provided lower rehabilitation costs for a railroad culvert and resulted in a renewed structure with every qualification for long, maintenance-free life.

The rehabilitation was accomplished by retaining the practical maximum of the old structure and completing it with new Prepakt Concrete. This formed a strong, integrated structural mass made possible by the low setting shrinkage and the high bonding strength of Prepakt to the original concrete.

The high quality of Prepakt Concrete insures this renewed railroad structure against deterioration due to severe temperature changes, flash floods, and other severe weathering conditions.

Full information on this and other Prepakt structures may be had by calling or writing the Main Office, Cleveland, Ohio.

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May 1952

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Centennial of Engineering 1952

Plans are well under way for the greatest assembly of engineers in history, the Centennial Convocation at Chicago, Ill., during the period September 3–13. The estimated attendance of 25,000 or more will represent some 50 engineering organizations in the United States and many in foreign countries.

In conjunction with the Centennial, ACI is planning its Regional Meeting September 10–12, 1952. Registration will begin on Wednesday, the 10th, and continue through Friday.

Wednesday will be open for attendance at the important events planned by the Centennial committee. Joint ACI-ASCE sessions will begin September 11. The first session, under the chairmanship of A. E. Cummings, will be on prestressed concrete. Contributions to this session are: Introductory paper bridging the gap between MIT's 1951 conference and this session, by Myle J. Holley, Jr., MIT; prestressed members of a New York harbor pier, by E. H. Praeger, Madigan-Hyland; the prestressed bridge program in Massachusetts, by J. C. Rundlett, Massachusetts Dept. of Public Works; prestressed concrete machine foundation, by A. M. Klein, Robert W. Hunt Co.; a discussion of needed research in prestressed reinforced concrete, by N. M. Newmark and C. P. Siess. Other possible papers include construction of a Caracas, Venezuela, bridge using the Freyssinet system, bridge construction using the Lee-McCall system, and prestressed girders.

The second session, to be held Friday, will deal with precast and thin shell concrete design and construction, Stewart Mitchell, chairman. Various aspects of thin shell design will be discussed by Bruce G. Johnston, University of Michigan. Construction aspects of thin shell structures is the topic of a paper by Anton Tedesko, Roberts and Schaefer Co. C. S. Whitney, Ammann and Whitney, will discuss construction of thin shell roofs. E. H. Praeger, Madigan-Hyland, will

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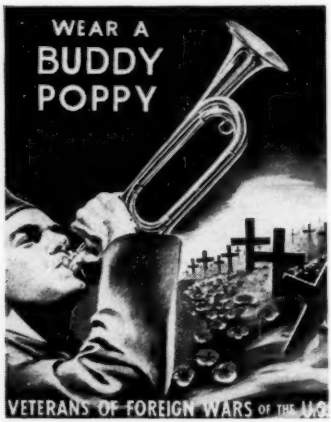
describe precast concrete boxes in a New York harbor pier. J. P. Thompson, PCA, will present a progress report on fire resistance of concrete floor slabs. Other probable subjects are: Outstanding structures in Madrid involving thin shell construction and precast concrete in highway bridge construction.

Full program details will be published in the June JOURNAL.

Program arrangements are being directed by C. E. Wuerpel, chairman, ACI Technical Program Committee, Marquette Cement Mfg. Co., Chicago, Ill. Other members of the committee are: L. H. Corning, PCA, Chicago; A. E. Cummings, Raymond Concrete Pile Co., New York, N. Y.; and Myle J. Holley, Jr., Massachusetts Institute of Technology, Cambridge, Mass.

H. C. Delzell, Concrete Reinforcing Steel Institute, Chicago, is chairman, ACI General Arrangements Committee; Howard F. Peckworth, American Concrete Pipe Assn., Chicago, is chairman, General Arrangements Coordinating Committee; and G. Donald Kennedy, PCA, Chicago, is chairman, Technical Program Coordinating Committee.

Members should plan to spend several extra days in Chicago to take in the many interesting events planned in connection with the Centennial.



ATTENTION!

The Conrad Hilton Hotel has a block of 200 rooms for ACI members attending the Regional Meeting and Centennial of Engineering in Chicago, Sept. 10, 11, 12, 1952. As all indications are that hotels will be crowded, ACI members are urged to make their plans and write for reservations as soon as possible. The Hotel cannot guarantee to accept reservations requested after August 10.

Write directly to Conrad Hilton Hotel, Chicago Ill., Attn: Mr. Edward Pelegrin, stating that reservations are requested in connection with the ACI REGIONAL MEETING.

Rates at the Conrad Hilton are as follows:

Single room:	$ 5.50—$12.00
Double room:	$ 9.50—$16.00
Twin-bed room:	$10.50—$16.00
Suites:	$20.00—$35.00

Symposium on airfield pavements for jet aircraft

The U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, Calif., was the scene of the Symposium on Airfield Pavements for Jet Aircraft, April 17–18, 1952.

Papers were contributed by representatives of many organizations including the Waterways Experiment Station, Corps of Engineers; Ohio River Division Laboratories, Corps of Engineers; U. S. Navy Bureau of Yards and Docks; and the Portland Cement Assn.

Soil stabilization conference

A three-day conference on soil stabilization will be held at MIT, June 18–20, 1952, to examine current problems in stabilization treatment.

Sponsoring the conference with MIT are the U. S. Navy Bureau of Yards and Docks, U. S. Army Corps of Engineers, the Highway Research Board, the Massachusetts Department of Public Works, and the U. S. National Council on Soil Mechanics and Foundation Engineering.

Who's Who This Month

Authors—Curing of Concrete

Six papers are presented beginning on p. 701 under the heading, "Curing of Concrete" by the following ACI convention participants:

A. G. Timms

A. G. Timms is materials engineer, Bureau of Public Roads, Washington, D. C. He started his professional career in 1919 as a part-time laboratory assistant in the Structural Materials Research Laboratory, Lewis Institute, under the direction of D. A. Abrams, obtaining a BS degree at Lewis Institute in 1920. He spent about a year in the construction field and from 1922 until 1926 he was employed on research projects in the laboratory of PCA.

In 1926 on leave of absence, he worked with the Immel Construction Co., Fond du Lac, Wis., as concrete technician on the construction of several buildings where one of the principal problems was the placing of concrete at low temperatures. From 1943–1946 he was engineer with the National Slag Assn., Washington, D. C., and in 1947 he joined the staff of the Bureau of Public Roads, Washington, D. C., as senior research engineer in the Concrete Research Laboratory.

D. L. Robinson

D. L. Robinson is senior engineer, Missouri State Highway Commission, Jefferson City, Mo. He received a BS degree in civil and irrigation engineering from the Colorado State Agricultural and Mechanical College in 1925 and was employed by the Missouri State Highway Dept. Bureau of Materials in 1926 where he has been engaged in general studies of cement and aggregate supplies and in the development of specifications for materials. Mr. Robinson has made studies of maintenance and construction costs of concrete pavement, flexible bases and surfaces and has investigated construction and maintenance procedures.

H. J. Gilkey

H. J. Gilkey, head, Department of Theoretical and Applied Mechanics, Iowa State College, Ames, Ia., has long been active in ACI affairs, both on technical and administrative committees. He is chairman of the Standards Committee and Committee 208, Bond Stress, as well as a member of Committee 115, Research. He is ACI's representative on the Highway Research Board and is active in ASCE, ASTM and SPEE.

In 1939 he received the ACI Wason Medal for the most meritorious paper. Professor Gilkey was a member of the Publications Committee, 1939–46; Board of Direction, 1937–38 and 1945–46; vice-president, 1947–48; and president, 1949.

Roy W. Carlson and W. R. Johnson

Roy W. Carlson, civil engineer, Berkeley, Calif., has been engaged mainly in making instruments for measuring stress, strain, pressure and temperature in concrete structures during the past twenty years.

Receiving a BA degree from the University of Redlands in 1922, he obtained his MS degree from the University of California in 1933 and DSc from Massachusetts Institute of Technology in 1939. He was associate professor of civil engineering at MIT until 1933 and at the University of California from 1935–36.

Dr. Carlson wrote several papers for the ACI JOURNAL until 1943 when he gave up concrete work for a number of years to work on atomic energy problems. Since World War II he has resumed work on concrete to a moderate degree, with emphasis on concrete dams. Dr. Carlson has been an ACI member since 1931.

W. R. Johnson has been with the Portland, Ore., District of the Corps of Engineers as senior materials engineer on concrete control since 1945. Projects of the Portland District include the Dorena, Detroit and Lookout Point Dams. He is a member of a committee set up by the Office, Chief of Engineers, to study the causes and methods of prevention of cracking in mass concrete structures.

After graduation from Lewis Institute, Mr. Johnson spent ten years in concrete research with the PCA Laboratory in Chicago. From 1934 to 1945 he was materials engineer with

TVA in charge of concrete on construction of Wheeler, Pickwick, Kentucky and Fontana Dams.

He was an ACI member from 1926 to 1932 and rejoined in 1934. He is active on ACI Committees 612, Recommended Practice for Curing Concrete, and 621, Aggregate Selection, Preparation and Handling.

G. E. Burnett

G. E. Burnett, acting assistant chief, Engineering Laboratories Branch, USBR, Denver, Colo., graduated from the University of Utah in 1930 and has been employed in the Engineering Laboratories Branch of the USBR since 1936. Much of his early work was devoted to investigation of sealing compounds for curing concrete. A white-pigmented sealing compound was prepared in the laboratory by Mr. Burnett in 1938 when he ground titanium dioxide pigment into a commercial grade of clear sealing compound with mortar and pestle. He had been head of the Bureau's paint laboratory from immediately following World War II until very recently when he was appointed to the position which he now holds.

Mr. Burnett has been a member of ACI since 1946 and is a member of ACI Committee 612, Recommended Practice for Curing Concrete, and chairman, Committee 616, Recommended Practice for the Application of Paint to Concrete Surfaces.

H. C. Vollmer

H. C. Vollmer, formerly research engineer, National Bureau of Standards, Washington, D. C., has had wide laboratory and field experience relating to both highway and structural concrete since 1930.

From 1930 to 1940 he was assistant to the materials engineer, Engineer Dept., District of Columbia Government. During World War II he was in charge of concrete control on the construction of the Pentagon for the U. S. Engineers. Subsequently he spent five years in the capacity of research associate on the curing and durability of portland cement concrete and correlating this work with field studies in New England, West Virginia and Delaware. Since leaving the National Bureau of Standards he has been with the Physical Research Laboratory of the Bureau of Public Roads as materials engineer in studies of cement and aggregates and presently is with the Dept. of the Army as a highway engineer.

Mr. Vollmer is an associate member of the Highway Research Board and is the author of several papers published in engineering periodicals.

Authors—Durability

Five papers which were presented at the ACI convention, are gathered together under the title of "Durability" starting on p. 725. Authors of these papers are:

C. B. Porter

C. B. Porter, assistant chief engineer, Chesapeake and Ohio Railway Co., Richmond, Va., is a graduate of the University of Cincinnati. In 1922 he entered railroad service as rodman on a field engineering party of the C&O RR. Subsequent service with the same railroad included instrument man, draftsman, resident engineer, assistant division engineer and division engineer. In 1944 he was appointed to his present position.

He is a member of AREA and past chairman, Subcommittee on Tunnels, AREA Committee 8, Masonry; vice-chairman, Committee 8, 1946-49; and chairman of the same committee, 1949-51. Mr. Porter is also a member of ASCE.

R. W. Gilmore

R. W. Gilmore is general bridge inspector, Western Region, B & O Railroad Co., Cincinnati, Ohio.

A graduate of the University of Cincinnati, he went to work for the Cincinnati, Hamilton and Dayton Railway (now B&O) in 1915 and has worked for the B&O ever since except for 18 months with Procter and Gamble and 12 months in the U. S. Army Signal Corps, Aviation Section.

His railroad work first consisted of construction and later track and structure work in the Maintenance of Way Department. For the last 17 years he has been engaged in bridge and structure maintenance.

F. H. Jackson

F. H. Jackson, physical research engineer, Bureau of Public Roads, Washington, D. C., has been active in ACI affairs since 1924. Past president of ACI, Mr. Jackson was awarded the ACI Wason Medal for the most meritorious paper in 1948. Associated with

Continued on p. 10

Union Oil Co. Laboratory, Brea, California. Masterplate "Iron-Clad" Concrete Floors—non-colored and colored. Equipment grouted with Embeco Non-Shrink Grout.

Builders and owners increasingly are taking the position that, because of their important effect upon plant operation, the floor and grout should be considered a part of the machine

In many such outstanding structures as this Union Oil Co. laboratory, Masterplate "Iron-Clad" Concrete Floors and Embeco Non-Shrink Grout are being specified and used because of their exceptional record of performance.

The long wear of the Masterplate Floor — 4-6 times greater than the best plain concrete floor — results from its thick, armored surface. This "iron-clad" surface, produced when the floor is installed or resurfaced, also makes the floor spark-resistant, corrosion-resistant, easy-to-clean, non-slip and economical.

Embeco helps overcome the problem of costly shutdowns by counteracting shrinkage — principal cause of failure in equipment grouts. Embeco non-shrink grout provides full level bed-plate contact, plus ductility to withstand impact, vibration and pounding action.

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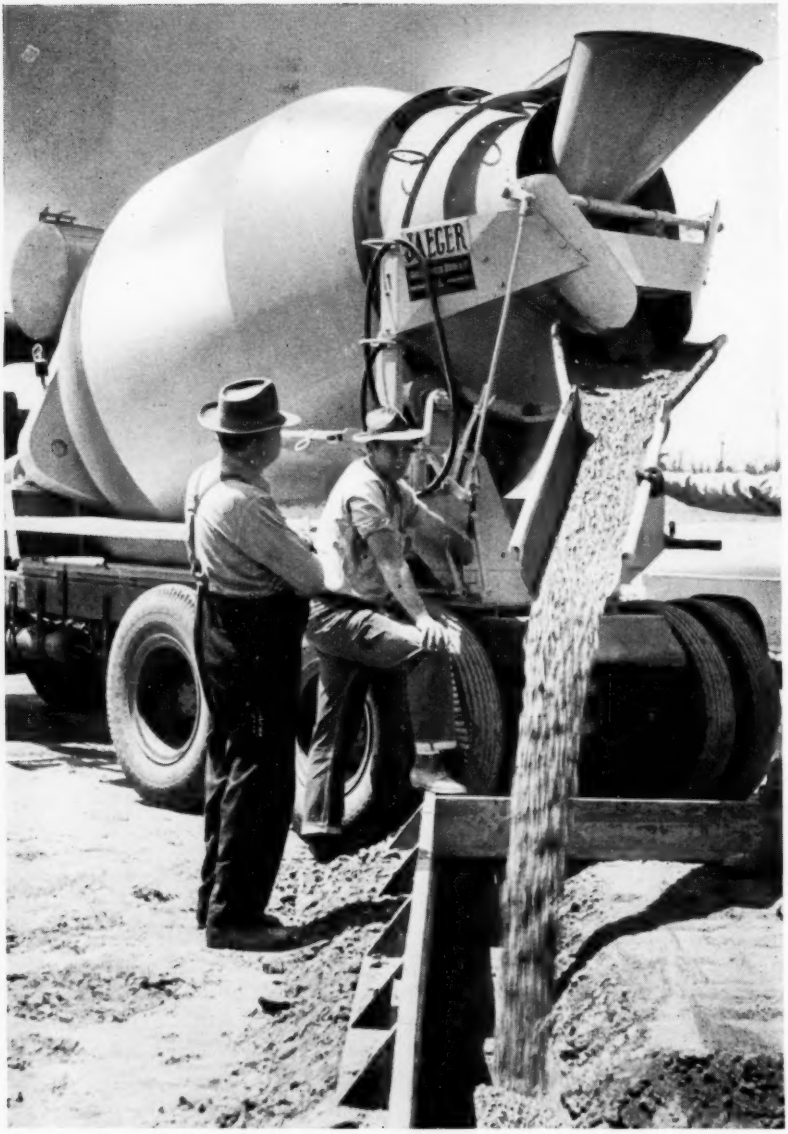
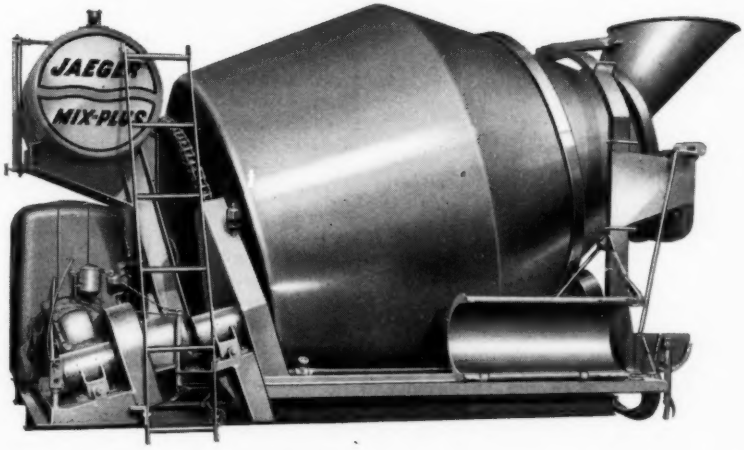


Because supervising architects and engineers recognize the quality of concrete produced by the Jaeger "dual mix" drum, more concrete is mixed in Jaeger truck mixers than by any other method.

Good concrete construction calls for properly mixed concrete . . .

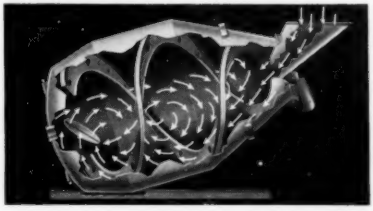


that's why Jaeger truck mixers never deviate from proven principles of "dual mix" drum design

Architects, engineers and contractors have long recognized the Jaeger "dual mix" drum as the most efficient ever developed for transit mixer service, which requires fast, end-to-end mixing in mass and clean, positive discharge without segregation, regardless of the slump.

Today, when many manufacturers are innovating truck mixers with the one object of hauling larger legal payloads, it is important to emphasize that Jaeger continues to adhere strictly to every proven principle of correct drum design.

In Jaeger's light weight "MIX PLUS" models you will find the same type of double-cone drum with the ratio of diameter to length necessary for proper mixing-in-mass. You will find the same continuous spiral mixing blades of precisely determined depth and contour and the same throw-back reversing blades (found in no other truck mixer), which enable the Jaeger drum to mix faster and discharge faster than any other.



Water control and distribution are equally efficient. Accurately metered water is delivered at the rate of 60 gpm at a positive pressure of 65 psi, through clog-proof rubber-sleeved jet which eliminates all possibility of grout entry. This extremely rapid, uniform and complete water distribution insures proper mixing of dry-batched materials even on shortest hauls.

These basic features, plus many operating improvements offered in Jaeger "Mix Plus" model truck mixers, are described in new Catalog TM-2. May we send you a copy?

THE JAEGER MACHINE CO., Columbus 16, Ohio

Mixers • Compressors • Pumps • Aggregate Spreaders • Concrete Spreaders and Finishers

Continued from p. 6

the Bureau of Public Roads since 1905, he has received recognition from several organizations for his work in the highway field, including the Distinguished Service Award of the Highway Research Board in 1948; the Meritorious Service Award of the U. S. Dept. of Commerce in 1950, and was elected to Honorary Membership in ASTM in 1948.

Lewis H. Tuthill

Lewis H. Tuthill, engineer, U. S. Bureau of Reclamation, Denver, Colo., has been an active ACI member since 1926, receiving the ACI Construction Practice Award in 1945 and serving as chairman of Committee 614 whose report on "Recommended Practice for Measuring, Mixing and Placing Concrete" became an ACI Standard in 1942.

Since graduation from Oregon State College in 1920, Mr. Tuthill has been continuously engaged in design and construction work on irrigation, water supply projects and the building of dams, with the technique and control of concrete as his primary and absorbing interest.

Since 1939 he has been a member of the staff of the Bureau of Reclamation, still pursuing the profession of concrete engineering and assisting in editing the Bureau's Concrete Manual. During a leave of absence from the Bureau, he spent several years during the war supervising concrete quality on the U. S. Maritime Commission's concrete ship construction program.

Byram W. Steele

Byram W. Steele, consulting engineer, Arlington, Va., recently retired after more than 15 years with the Office, Chief of Engineers, Washington, D. C.

An ACI member since 1929 he is active on ACI Committees 212, Admixtures; 620, Construction Joint Practice; and 621, Aggregate Selection, Preparation, Handling and Use.

For 16 years he was with the Bureau of Reclamation in charge of storage dam design and materials investigations. Economical, durable concrete for hydraulic structures has been a specialty with Mr. Steele since the construction of Hoover Dam when the first extensive mass concrete investigations were undertaken.

Chuzo Itakura

Chuzo Itakura, Faculty of Technology, Hokkaido University, Japan, presented his paper, "Electric Heating of Concrete in Winter Construction," p. 753, at the ACI convention.

A graduate of Hokkaido University in 1932, he was engaged in design, supervision and performance of field works, especially winter construction, for more than three years. Since 1935, as assistant professor at Hokkaido University, he has been devoted to research in freezing damage of materials and structures and cold weather construction. In 1946 he became a doctor of engineering and a professor.

Dr. Itakura has written a number of papers and reports in Japanese relating to freezing damages, electric heating of concrete, and lightweight concrete and its construction.

He has been an ACI member since 1951.

Clair L. Johnson

"Field Problems in Constructing a Prestressed Concrete Bridge," p. 761, was presented at the ACI convention by Clair L. Johnson, Johnson and Anderson, Engineers, Pontiac, Mich.

Mr. Johnson received a BS degree in civil engineering from Michigan State College in 1933. Since graduation he has been engaged in general engineering, principally municipal improvement and development.

After release from the U. S. Army in 1946 he formed the firm of Johnson and Anderson, Engineers, with Felix A. Anderson. They began working with prestressed concrete in 1949 and have since formed Anjoh, Inc., to promote and develop use of prestressed concrete for building and bridge use.

Mr. Johnson joined ACI in 1951.

Herbert A. Sawyer, Jr.

Herbert A. Sawyer, Jr., associate professor of civil engineering, University of Connecticut, Storrs, Conn., is the author of "Economy of Concrete Beams," p. 773, which was presented at the ACI convention.

Professor Sawyer attended Drew University for two years and received a BS in CE from Lafayette College in 1939. Recently he received a professional degree of CE from the same university. He received his Masters degree from Yale University where he was a part-time instructor.

Professor Sawyer was employed by the Chicago Bridge and Iron Co. at Chicago, Ill., Morgan City, La., and Birmingham, Ala., from 1941–46. During most of this time he was engineer in charge of construction of floating dry docks for the Navy for which he received the Meritorious Civilian Service Award of Budocks.

In 1946 he returned to teaching courses in structural engineering at Lehigh University. He taught at the University of Alabama and the University of California for a short time. Since 1950 he has been at the University of Connecticut.

Professor Sawyer joined ACI in 1949. He is also a member of ASEE, AAUP, ASCE, Tau Beta Pi, Phi Beta Kappa, Sigma Xi and Chi Epsilon. He is a registered professional engineer in Alabama and is author of several professional papers and discussions.

William K. Hatt

William K. Hatt, ACI member since 1910, died recently. Dr. Hatt was educated at the University of New Brunswick receiving a BA degree in 1887, MA degree in 1898 and PhD in 1901. He also received a civil engineering degree from Cornell University in 1891.

He began his teaching career as a professor of civil engineering at the University of New Brunswick in 1891–2, followed by a year at Cornell University. In 1893 he joined the faculty of Purdue University. In his 45 years at Purdue he taught applied mechanics and civil engineering, since 1907 as Professor of Civil Engineering and Director of the Laboratory for Testing Materials.

He served on many state and national commissions on work involving conservation, flood prevention, highway research and building codes and was the author of numerous articles published in the ACI JOURNAL as well as many other engineering magazines.

Dr. Hatt was ACI President from 1917–19. He received the Fuertes Gold Medal from Cornell University in 1903 and the ACI Turner Medal in 1929 for "pioneer work in reinforced concrete research." In 1932 he was elected to Honorary Membership in ACI and in 1938 was elected to Honorary Membership in ASTM.

In addition to ACI and ASTM, Dr. Hatt was affiliated with AREA, Society for the Promotion of Engineering Education, Sigma Xi, Tau Beta Pi and Chi Epsilon.

G. L. Lindsay

G. L. Lindsay, director of tests and research, Universal Atlas Cement Co., New York, N. Y., died recently.

Mr. Lindsay had been an ACI member since 1929 and was elected to its Board of Direction in 1951.

Before and after entering college, he worked during several vacation periods at Universal's Duluth plant on construction and in the physical laboratory. After receiving a BS degree in chemical engineering from the University of Minnesota in 1921, he rejoined Universal Atlas Cement Co. as a chemist at Duluth, Minn., and in 1925 moved to the main office then at Chicago, where he served successively as engineer in the Inspection Bureau and as engineer of tests. In the latter capacity Mr. Lindsay came to New York when the company moved its main office in 1939. He was promoted to assistant director of tests and research in 1943 and in the following year was appointed director of tests and research.

In addition to his ACI membership, Mr. Lindsay was a member of ASTM. He was active on ASTM Committee C-1 as well as several subcommittees of that committee.

Eric Plagwit

Eric Plagwit, ACI member since 1921, died recently. Mr. Plagwit had been active in the engineering field for nearly 54 years and had been head of the chimney division of Rust Engineering Co. since 1924.

A native of Berlin, Germany, he came to America in 1897 as a mechanical engineer after receiving his degree at the University of Charlottenburg in Berlin. At the time of his death he was active on chimney projects in Australia, Canada, Chile, Mexico and Uruguay.

In addition to his affiliation with ACI, he was a member of ASME.

C. L. Warwick

C. L. Warwick, executive secretary, ASTM, and its administrative head since 1919, died suddenly, April 23.

Mr. Warwick had been active in the Society since 1909 when he graduated in civil engineering from the University of Pennsylvania. While he was an instructor and assistant professor at the University, he also served as assistant secretary of ASTM with Edgar Marburg, the Society's founder-secretary. In 1919 on the death of Dr. Marburg he was appointed secretary-treasurer (chief executive officer) and in 1946 became executive secretary.

He had made many notable contributions to the field of standardization and research in materials and was recognized as an outstanding authority on materials. One of his important contributions was in the War Production Board during World War II, where he served as head of the Specifications Branch of the Conservation Division and later headed the Materials Division.

In addition to being a member and former vice-president of the Engineers Club of Philadelphia, Mr. Warwick was affiliated with ASCE, American Association for Advancement of Science, American Society for Metals, Sigma Tau and Sigma Xi.

Traffic engineering course

A one-week course in traffic engineering will be presented at the University of California, Berkeley, June 23–27, 1952. Being arranged by the University's Institute of Transportation and Traffic Engineering, the course will provide intensive coverage of fundamentals and current problems. Some of the major subjects to be discussed are orientation in traffic engineering; characteristics of traffic volumes, speeds and congestion; engineering for accident prevention; highway capacity; geometric design; and traffic engineering administration.

Cooperating with the Institute in presenting the course are the California Division of Highways, the League of California Cities and the Western Section, Institute of Traffic Engineers.

Correction

The building shown on the March *News Letter* cover should have been identified as the General Mills, Inc., cereal plant at Lodi, Calif. Precast Mosai panels were fabricated by the Otto Buehner Co., Salt Lake City, Utah. Ned H. Abrams, Sunnyvale, Calif., was the architect.

East and west coast conferences on effects of atomic bombs and earthquakes on structures

Conferences on the effects of atomic bombs and earthquakes on structures are slated for June on opposite sides of the United States. The University of California offers a symposium on "Earthquake and Blast Effects on Structures," June 26–28, and the Massachusetts Institute of Technology presents a conference on "Building for the Atomic Age," June 16–17.

The symposium is being sponsored jointly by the Earthquake Engineering Research Institute and the Department of Engineering, University of California, to make available to those engaged in planning, design and construction of structures the latest research results and professional developments in the field of dynamic loading of structures due to effects of earthquakes and blast.

Various papers included on the program are "Motion of Earthquakes," "Forces on Structures Due to Blast," "Analysis of Response of Structures to Dynamic Loading," "Failure Observations," "Building Code Provisions for Aseismic Design," and "Design for Blast Loading."

An exhibit of modern instruments for detecting and recording dynamic phenomena is planned and will be on display throughout the symposium.

Further information and a copy of the final program may be obtained from the Department of Conferences and Special Activities, University of California Extension, Los Angeles 24, Calif.

The conference arranged by the Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, will present the most recent knowledge on the effects of atomic bombs and the design of structures for resistance to large blasts.

A few of the papers to be presented are: "General Effects of Atomic Bombs," "General Reaction of Buildings," "Reinforced Concrete Structures," "Precast Concrete Structures," and "Cost of Blast-Proof Structures."

Further information, a detailed program, and a registration application form may be obtained from Summer Sessions Office, Massachusetts Institute of Technology, Cambridge 39, Mass., Attention: Building in the Atomic Age Conference.

Positions and Projects — ACI Members

Blanks appointed research consultant

Robert F. Blanks has been appointed research consultant to the Ideal Cement Co., Denver, Colo. This appointment is in addition to his capacity as vice-president and general manager of Great Western Aggregates, Inc., a subsidiary of Ideal Cement Co.

Sommerschield heads engineering division

Harold F. Sommerschield recently became associated with Abell-Howe Co., engineers and contractors, to head their new engineering division. He was previously structural field representative for PCA.

McHenry appointed PCA administrative assistant

Douglas McHenry, formerly head of the Concrete Laboratory Section, USBR, Denver, Colo., has been appointed administrative assistant to A. Allan Bates, vice-president for research and development, PCA.

Mr. McHenry is known in engineering circles as the author of numerous articles on concrete technology, and in particular for his work while head of the Structural Research Section of the Engineering Laboratories Branch of the USBR, a position which he held for nine years prior to his appointment as head of the Concrete Laboratory Section in September 1951.

Before joining the staff of the Bureau of Reclamation in 1940, he served for six years with TVA on the construction of Norris and Hiwassee Dams and power plants, and on research studies of the structural behavior of TVA structures.

Mr. McHenry has been an ACI member since 1936 and is chairman of ACI's Technical Activities Committee.

Jones and Wood retire

Paul A. Jones and Ernest B. Wood recently retired from the USBR's Design and Construction Division, Denver, Colo.

Mr. Jones, who was chief of the Administrative Branch—Construction, has a total of 38 years with the Bureau. His work included such projects as Hoover and Hungry Horse dams.

Mr. Wood joined the Bureau in 1935 as a member of the Engineering Laboratories where he was in charge of the preparation of specifications for concrete and other types of construction materials.

Mr. Jones, ACI member since 1938, also is a member of ASCE. Mr. Wood, affiliated with ACI since 1936, is a life member of the Colorado Society of Engineers.

ACI members honored

Emil A. Gramstorff, chairman, department of civil engineering, Northeastern University, Boston, Mass., was elected president of the Boston Society of Civil Engineers at its 104th annual meeting held last March. Following the election of officers, awards were presented. The Desmond Fitzgerald Medal was presented to Prof. John M. Biggs, MIT, and Prof. Myle J. Holley, Jr., MIT, received the Structural Section Award.

Grinter appointed dean

Dr. L. E. Grinter, formerly vice-president of Illinois Institute of Technology, has been appointed dean of the graduate school and director of research, University of Florida, Gainesville, Fla. At the University he will assume responsibility for a graduate student body of 1500 students and a research program involving an expenditure of several million dollars annually. He expects to take up his new duties at the beginning of the fall semester.

For the past two years he has been consultant to the Board of Control for Southern Regional Education and is now a member of the Board's Commission on Graduate Studies which has its headquarters in Atlanta, Ga. Dr. Grinter has been an ACI member since 1938.

PROFESSIONAL CARD

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Ohrt resigns

Frederick Ohrt has resigned as manager and chief engineer of the Honolulu Board of Water Supply after nearly 27 years as head of the system. He resigned after a Honolulu court had appointed him to a lifetime job as one of three trustees of the James Campbell estate.

Higginson heads USBR concrete lab

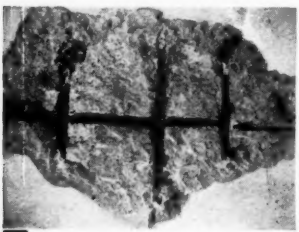
Elmo C. Higginson, formerly assistant to chief, Engineering Laboratories Branch, Bureau of Reclamation, Denver, has been appointed head of the concrete laboratory of the same organization.

Construction offers opportunity to civil engineering graduates

Construction Methods and Equipment, a McGraw-Hill publication, maintains that civil engineering graduates should take a good look at construction if they are seeking an interesting and lucrative career. The magazine says that construction, the end-product of civil engineering, totaled $39 billion last year. The graduate who wants to learn the business can start as a timekeeper, material checker, assistant job engineer or even a laborer, the magazine further points out.

Clair honored

Miles N. Clair, president, Thompson and Lichtner Co., Inc., Consulting Engineers, Brookline, Mass., recently was paid high tribute by the ASCE student chapter of Northeastern University, when this group made him its first honorary member. According to Prof. E. L. Spencer of the University, Mr. Clair has given unstintingly of his time and energy for the past decade in serving as senior contact member with Northeastern students.



Typical example of spalling. Note corrosion of reinforcing rods exposed by disintegrating concrete.

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Honor Roll

February 1—March 31, 1952

Newlin D. Morgan, Sr., leads with 26 credits; Walter H. Price is second with six credits; Joseph A. Brunner, George C. Ernst and Arturo Obadia B. are tied for third place with four credits each.

Is your name on this year's Honor Roll? Are YOU Member conscious?

Newlin D. Morgan, Sr. (Ill.)	26
Walter H. Price (Colo.)	6
Joseph A. Brunner (N. Y.)	4
George C. Ernst (Nebr.)	4
Arturo Obadia B. (Venezuela)	4
F. Thomas Collins (Calif.)	3
Theodore O. Reyhner (Colo.)	3
Howard Simpson (Mass.)	3
Pedro Arismendi A. (Venezuela)	2
J. R. Cassell (Md.)	2
Raymond A. Davis (Calif.)	2
L. G. Farrant (Fla.)	2
Hunter W. Hanly (Ohio)	2
Lane Knight (Canada)	2
I. E. Morris (Ga.)	2
I. S. Rasmusson (Colo.)	2
Rafael Rodriguez (P. R.)	2
Mills Sampey (Ohio)	2
Saul Shaw (N. J.)	2
Chester P. Siess (Ill.)	2
G. B. Southworth (Ohio)	2
Willard H. Stevenson (Ind.)	2
Henry Wilckens (Australia)	2
Anis Yarid (French Morocco)	2
Robert Zaborowski (N. Y.)	2

List four more "25ers"

A few months ago a list of "The 25's" was published in the JOURNAL* —men who from 1925 to 1950 sponsored 25 or more ACI Members. Appropriate note was made, too, in the alphabetical section of the 1951 ACI Directory.

R. H. Sherlock became the 26th Member-Getter a year ago and now *Edward W. Thorson, Jose Antonio Vila* and *Jay E. Jellick* have joined the ranks.

Office files show some gains were made slowly over the years, others suddenly; several lack but a few points of making the grade. Perhaps they and others will be inspired to increase their efforts during the coming year.

*Sept. 1951 *News Letter*, p. 8.

New Members

The Board of Direction approved 64 Individual, 1 Contributing, 3 Corporation, 12 Junior and 27 Student Memberships for March. With adjustments for losses—resignations, deaths, nonpayment—the total membership on April 1 was 5677.

Individual

ALDERMAN, BISSELL, Holyoke, Mass. (Arch.)
BATNI, K. B., New Delhi, India (Director, Road Research Institute)
BOWEN, GERALD, Los Angeles, Calif. (Pres., Presan Corp.)
BRUNETTE, L. S. C., Salisbury, S. Rhodesia (Managing Dir., L. S. Brunette, Ltd.)
BURRUSS, ELMER E., Charlottesville, Va. (Arch.)
CAMPBELL, ROBERT F., San Gabriel, Calif. (Chf. Engr. & Assoc., F. Thomas Collins & Assocs.)
CAYTING, FRANK J., Oakland, Calif. (Sales Mgr., Rhodes & Jamieson, Ltd.)
CHUBBUCK, WALTER C., Canyon Ferry, Mont. (Field Inspection, USBR)
COLLINS, RADLEY ELLIOT, Hartsdale, N. Y. (Staff Engr., Johns-Manville Corp.)
CONOVER, J. C., Cincinnati, Ohio (Super. of Inspection, The H. C. Nutting Co.)
COOLEDGE, VICTOR R., Berkeley, Calif. (Engr., Southern Pacific Co.)
CRIMM, R. G., Havertown, Pa. (Branch Office Mgr., Master Builders Co.)
DAVIS, REES L., Casper, Wyo. (Lab. Testing, USBR)
DUNN, HOWARD C., Columbia Falls, Mont. (Matls. Engr., USBR)
DUX, HERBERT E., Hancock, W. Va. (Chf. Engr., Pa. Glass Sand Corp.)
FEENEY, DAVID J., Kilburn, S. Australia (Mgr., Orlit (S.A.), Ltd.)
FINK, ALBERT A., Antioch, Calif. (Struct. Engr., Fibreboard Products, Inc.)
GARROD, DONALD E., Marion, Ohio (Office, Field, Floyd G. Browne & Assocs.)
GILES, JAMES HOWARD, JR., Washington, D. C. (Matls. Engr., Cement Reference Lab., National Bureau of Standards)
GREENE, LEROY F., Sacramento, Calif. (C.E.)
HAMMASSANI, HASSEIN EL, Cairo, Egypt (Engr., Ministry of Public Works)
HANSON, ARTHUR C., Austin, Minn. (Asst. City Engr., City of Austin)
KONCZA, L., Chicago, Ill. (Student, Univ. of Illinois)
LANCE, WILLIAM J., San Antonio, Texas (Arch. & Engrg. Research, Southwest Research Institute)
LARRABEE, D. M., Waterloo, Ia. (City Engr., City of Waterloo)
LARSON, BENNETT C., Chicago, Ill. (Struct. Design, Dist. Public Works Office, Ninth Naval Dist.)
LEULIETTE, ELMER, New York, N. Y. (Sales Engr., Universal Concrete Pipe Co.)
MATHIESON, ROBERT P., Pasadena, Calif. (Estimator, Ecker Bros.)
McCONNELL, CLIFFORD HARVEY, Fort Peck, Mont. (Chief, Constr. Div., Corps of Engrs.)
McCUNE, R. F., Denver, Colo. (Vice-Pres., General Mgr., J. L. Peterson, Inc.)
McLEOD, JOHN R., Shawinigan Falls, Canada (Concrete Tech., Shawinigan Engrg. Co., Ltd.)
McQUISTION, ALBERT H., Cleveland, Ohio (Struct. Design, Wilbur Watson Assocs.)
MEIER, JAMES E., Cincinnati, Ohio (Engr.)
MELLINGER, FRANK M., Cincinnati, Ohio (Director, Ohio River Div., Corps of Engrs.)
MOORE, GEORGE W., Vancouver, Canada (Salesman, The Master Builders Co.)
MOYSE, JACK, Louisville, Ky. (Design, Super., Thomas J. Nolan & Sons)
MUSKIN, JACOB C., New York, N. Y. (Chf. Struct. Engr., Singmaster & Breyer)
NELSON, HARRY C., Seattle, Wash. (Struct. Design, Rotla Sheet Metal Works, Inc.)
NELSON, ROBERT G., Seattle, Wash. (Constr. Inspector, Corps of Engrs.)

NEUBERT, CARL, Buffalo, N. Y. (Struct. Engr., The Austin Co.)
OKADA, KIYOSHI, Kyoto, Japan (Asst. Prof., Kyoto Univ.)
OLSON, GEORGE G., Phoenix, Ariz. (Research Dir., Builders Supply Corp.)
ORTIZ, PEDRO JUAN, San Juan, P. R. (Engr.)
PARRAVANO, WILLIAM F., Cleveland, Ohio (Contr.)
PETT, JOHN F., Westfield, N. J. (Hydraulic Engr., Ebasco International Corp.)
PUJALS, AMADEO G. MIJARES, Bayamo, Cuba (Public Works Dept., Govt. of Cuba)
REYNOLDS, W. ARTHUR, Belmont, Mass. (Instructor, Wentworth Inst.)
ROGGENBACH, DEFORREST R., Lincoln, Nebr. (Struct. Design, Clark & Enersen)
SAAD, MICHEL A., Aleppo, Syria (Prof., College of Engrg.)
SARDI S., VICTOR, Caracas, Venezuela (C.E.)
SAWYER, DAVE H., Lexington, Ky. (Solvay Sales Div., Allied Chem. & Dye Corp.)
SCHOENROCK, KEITH G., Garretson, S. D. (Contr.)
SHAFFER, ROBERT W., Los Angeles, Calif. (Exec. Vice-Pres., Presan Corp.)
SKILLMAN, ELMER I., Denver, Colo. (Matls. Engr., USBR)
SMITH, GEORGE F., Los Angeles, Calif. (Chf. Struct. Engr., Koebig & Koebig, Inc.)
SPRONCK, MICHAEL A., New York, N. Y. (Assoc. Editor, Contractors & Engrs. Monthly)
STOWE, HERMAN M., Tacoma, Wash. (Constr. Engr., Bonneville Power Admin.)
SWAN, ROBERT G., Los Angeles, Calif. (Contr.)
SWEETON, ARTHUR W., III, Hartford, Conn. (Chf. Design Engr., Bureau of Public Works)
THORNTON, RUTH C., Birmingham, Ala. (Librarian, Birmingham Public Library)
TRINKO, LOUIS E., Urbana, Ill. (Student, Univ. of Ill.)
WARNER, LOUIS ARTHUR, E. Haven, Conn. (Engr., Westcott & Mapes, Inc.)
WOLF, WALTER W., Chicago, Ill. (Sales Engr., Besser Mfg. Co.)
WOODWARD, JONES D., Jackson, Miss. (Engr., Drafting, Jackson Stone Co.)

Contributing

ANDERSON, MACGREGOR S., Houston, Texas (Cons. Engr., Southwest Dist. Mgr., Univ. Form Clamp Co.)

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OFICINA TECNICA GUTIERREZ & CO., S. A., Caracas, Venezuela (Carlos E. Daboin)

Junior

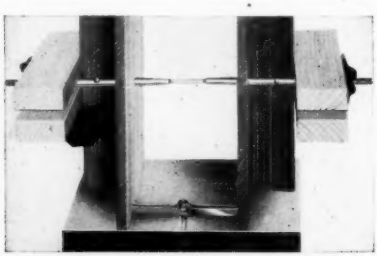
ALBRECHT, THEODORE J., JR., St. Louis, Mo. (Corps of Engr.)
BARSAMIAN, K. VAROUJAN, Beirut, Lebanon (Design, Drafting, Supervision, Trans-Arabian Pipe Line Co.)
BASSI, KRISHAN GOPAL, London, England (Student, Imperial College of Science & Tech.)
DINOS, COSTANTI, Corozal, P. R. (Struct. Design, Schimmelpfennig-Ruiz-Gonzalez-Munoz Marin)
DOLYNIUK, ANDREW K., Belfield, N. D. (Inspection, USBR)
EAGLE, PAUL, Bronx, N. Y. (Design, Drafting, Byrne Assocs.)
FREIHOFER, CHARLES F., Indianapolis, Ind. (Field Inspection, Toledo Testing Lab.)
MAGUIRE, DOUGLAS LOWELL, Regina, Canada (Design, Drafting, Inspection, Waterworks Dept.)
MCCLAVE, JAMES C., Cliffside, N. J. (McClave & McClave, Designer, Drafting)
PERSONS, GEORGE F., Elyria, Ohio (Lab. Testing, West Side Lumber & Coal Co.)
PODOLNY, WALTER, JR., Cleveland, Ohio (Design, Drafting, Wilbur Watson Assoc.)
REICHEL, GILBERT, Brooklyn, N. Y. (Struct. Design, Brown & Blauvelt)

Student

BENNETT, RUSSELL, Durand, Ill. (Univ. of Ill.)
CUFAUDE, RALPH E., Champaign, Ill. (Univ. of Ill.)
DUFER, HARLAN H., Champaign, Ill. (Univ. of Ill.)
EINSWEILER, ROBERT, Champaign, Ill. (Univ. of Ill.)
EKMAN, KENNETH WILLIAM, Wilmette, Ill. (Univ. of Ill.)
ERBACH, JAMES EDWARD, Chicago, Ill. (Univ. of Ill.)
FEARS, DONALD J., Champaign, Ill. (Univ. of Ill.)
FRAZIER, JOHN A., Champaign, Ill. (Univ. of Ill.)
HENDERSON, JAMES A., Springfield, Ill. (Univ. of Ill.)
JACHEC, STANLEY W., Wauconda, Ill. (Univ. of Ill.)
JOHNSTON, ALBERT S., Champaign, Ill. (Univ. of Ill.)
KING, JOHN MORROW, Springfield, Ill. (Univ. of Ill.)
JAYACHAND, KUMBHANI RAJNIKANT, Chicago, Ill. (Ill. Inst. of Tech.)
LEE, DOUGLAS HERBERT, Champaign, Ill. (Univ. of Ill.)
LINDAHL, JOHN MATTHEW, Chicago, Ill. (Univ. of Ill.)
MAINA, RONALD A., Chicago, Ill. (Univ. of Ill.)
MARTIN, JOHN D., Champaign, Ill. (Univ. of Ill.)
MEYER, NORMAN H., Lorraine, Kans. (Univ. of Ill.)
NEWMAN, EDWARD I., Denver, Colo. (Univ. of Denver)
PRATT, HARLAN A., Champaign, Ill. (Univ. of Ill.)
ROJAS-ROHL, CARLOS, Lansing, Mich. (Mich. State College)
SAEZ, TOMAS, Baton Rouge, La. (La. State Univ.)
SEITZ, ARTHUR M., Champaign, Ill. (Univ. of Ill.)
SWENDSON, ALFRED S., Greendale, Wis. (Univ. of Ill.)
TYSON, J. WALTER, Champaign, Ill. (Univ. of Ill.)
WEAR, JOHN ROSS, Danville, Ill. (Univ. of Ill.)
WEISS, SUMNER B., Hartford, Conn. (Univ. of Ill.)

Miller and Blackwood elected vice-presidents

T. T. Miller and George W. Blackwood have been elected vice-president—marketing and vice-president—general sales manager, respectively, Dewey and Almy Chemical Co., Cambridge, Mass.



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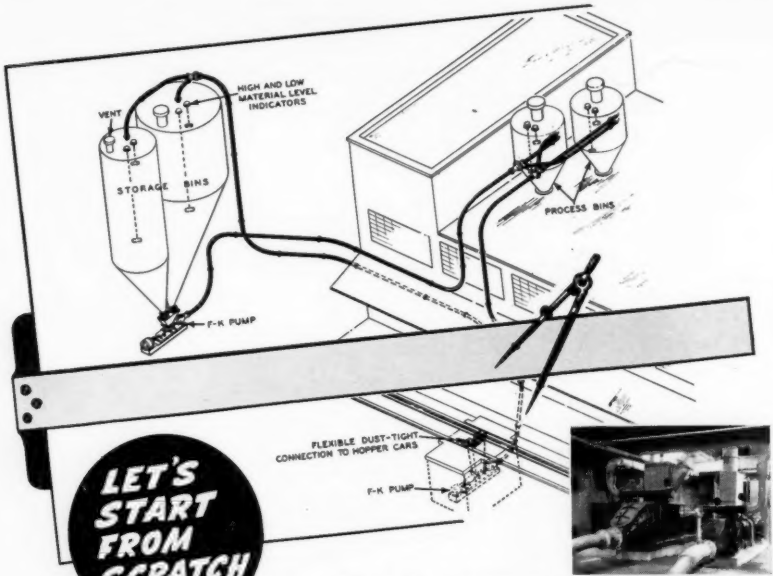

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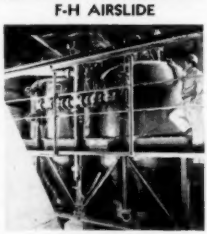

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ACI Technical Committees

Technical Activities Committee

Organized 1947

The Technical Activities Committee is responsible, under Board of Direction, for technical publications, convention programs and technical committee activities (see p. 23 of 1951 ACI Directory). Terms of members expire at the annual convention of the year indicated. The President of the Institute is a member during and by reason of his tenure of office; similarly the Secretary-Treasurer of the Institute is secretary of the committee. The chairman, president and secretary constitute the committee's executive group.

Douglas McHenry, (1954) Chairman
A. T. Goldbeck, (Ex Officio)
Fred F. Van Atta, Secretary
Herbert Insley (1953)
Bryant Mather (1954)
R. W. Spencer (1953)
Myron A. Swayze (1953)
L. W. Teller (1954)
I. L. Tyler (1954)

Standards Committee

Organized 1937

The Standards Committee supervises the promulgation of standards according to rules adopted by the Board of Direction (see p. 24 of 1951 ACI Directory).

Herbert J. Gilkey (1954) Chairman
Harmon S. Meissner (1953) Vice-Chairman
Fred F. Van Atta, Secretary
Frank H. Jackson (1953)
Clyde T. Morris (1954)
John R. Nichols (1953)
Douglas E. Parsons (1954)
Stanton Walker (1954)

Committee 115—Research

Organized 1921

This committee was organized to affiliate the interests of research workers and agencies in the field of concrete. Its assignment is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives. A compilation of research projects is prepared and distributed each year at the open session of the committee at the annual convention. During the past year 62 organizations from all parts of the United States and several foreign countries reported to the committee. They included universities, state and federal highway departments, cement companies, industrial and commercial laboratories and associations; reported 396 different projects underway, covering research in the fields of aggregates, admixtures, cements and plain and reinforced concrete.

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Ohio River Div. Laboratories, Dept. of the Army
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Virginia Polytechnic Institute
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University of Iowa
WARREN RAEDER
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R. I. ROWELL*
Vermont State Highway Dept.
CHARLES H. SCHOLER
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J. R. SHANK
Ohio State University .
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KENNETH B. WOODS
Purdue University
CHARLES E. WUERPEL
Marquette Cement Manufacturing Co.
RODERICK B. YOUNG
Hydro-Electric Power Com. of Ontario

Committee 207—Properties of Mass Concrete
Organized 1930

Committee 207 is responsible for reporting developments in mass concrete construction. Two task groups appointed in 1951, one to prepare a report on properties of mass concrete and the other to give consideration to the mass of data compiled by the former Committee 207 regarding the long-time behavior of mass concrete structures in service, are expected to report to the committee in the near future.

ROBERT F. BLANKS, Chairman
Bureau of Reclamation
FEDERICO BARONA DE LA O
Bureau of Hydraulic Resources, Mexico
RAYMOND E. DAVIS
University of California
JEROME M. RAPHAEL
Bureau of Reclamation
I. L. TYLER
Portland Cement Assn.
WILLIAM R. WAUGH
Office, Chief of Engineers, Dept. of the Army
RODERICK B. YOUNG
Hydro-Electric Power Com. of Ontario

Committee 208—Bond Stress
Organized 1930

This committee is assigned to the promotion of research and to the collection and interpretation of data from field and laboratory to determine proper working stresses for bond in relation to type of bar employed in reinforced concrete construction. With regard to design matters the role of Committee 208 is purely advisory. The task immediately ahead is a continuation of current activity: to scrutinize the data and to inaugurate new recommendations or modify existing ones as the need becomes apparent.

HERBERT J. GILKEY, Chairman
Iowa State College
ARTHUR P. CLARK
AISI Fellow, National Bureau of Standards
H. F. GONNERMAN
(CARL A. MENZEL, alternate)
Portland Cement Assn.
W. H. JACOBS
Rail Steel Bar Assn.
RAYMOND C. REESE
Consulting Engineer
CHESTER P. SIESS
University of Illinois
DAVID WATSTEIN
National Bureau of Standards
A. C. WEBER
Laclede Steel Co.

Committee 209—Volume Changes and Plastic Flow in Concrete
Organized 1930

This committee, reorganized in 1946, consolidates the previous assignments of Committee 102, Volume Changes in Concrete, and Committee 109, Plastic Flow. The subjects proved to be virtually inseparable and the conclusions and implications of research in these two subjects will

*Non-Member ACI.
†Corporation Member.

be reported by the present committee. Bureau of Reclamation long-time tests, and others, are being studied by the committee with a view to presentation of a paper for publication.

DOUGLAS MCHENRY, Chairman
Portland Cement Assn.
HARMER E. DAVIS
University of California
GERALD PICKETT
Kansas State College
CLARENCE RAWHOUSER
Foreign Activities Office, Bureau of Reclamation
WILLIAM H. THOMAN
University of Colorado

Committee 210—Resistance to Erosion in Hydraulic Structures

Organized 1940

The committee is collecting information which will enable it to publish a report of benefit to designers and specification writers concerned with hydraulic structures. A new draft is in preparation with a view to publication before the end of the year.

WALTER H. PRICE, Chairman
Bureau of Reclamation
R. R. CLARK
Corps of Engineers, Dept. of the Army
JACOB J. CRESKOFF
Consulting Engineer
W. T. MCCLENAHAN
Sanitary District of Chicago
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GEORG WASTLUND
Swedish Cement and Concrete Institute
RODERICK B. YOUNG
Hydro-Electric Power Com. of Ontario

Committee 212—Admixtures

Organized 1943

This committee studies, assembles and reports information on the effect of various admixtures, including air-entraining agents, on the properties of concrete. Admixtures are defined by the committee as "a substance other than portland cement, aggregate, or water that is used as an ingredient for concrete." The committee is launched on a study to report the advantages and limitations of admixtures in (1) grouting operations, (2) cement-alkali aggregate reactions, (3) grading deficiencies, (4) concrete products, (5) improvement in resistance to freezing and thawing, (6) underwater construction, (7) "false set," (8) as integral waterproofers. Final editing of a report is in progress, aimed at making it available for early publication.

*Non-Member ACI.

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National Bureau of Standards
FRANK H. JACKSON
Bureau of Public Roads
ALEXANDER KLEIN
University of California
T. C. POWERS
Portland Cement Assn.
BYRAM W. STEELE
Consulting Engineer
CHARLES E. WUERPEL
Marquette Cement Manufacturing Co.

Committee 213—Properties of Lightweight Aggregates and Lightweight Aggregate Concrete

Organized 1946

This committee's assignment is to gather, correlate and report available information on the properties of concrete made with mineral lightweight aggregates in the fields of monolithic structural concrete, precast concrete, and insulating and roof and floor fill concrete, with special reference to the following: unit weight, compressive strength, transverse strength, absorption, permeability, resistance to freezing and thawing action, resistance to high temperatures, volume changes due to moisture variations, elastic and plastic properties. The committee will confine first report to considerations of available commercial aggregates. Preliminary work is under way.

GEORGE W. WASHA, Chairman
University of Wisconsin
J. JOHN BROUK
Precast Slab and Tile Co.
CLIFTON C. CARLSON
Portland Cement Assn.
JACK E. COUNTS
Anchor Designs
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University of Florida
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A. G. TIMMS
Bureau of Public Roads
LEWIS H. TUTHILL
Bureau of Reclamation
H. T. WILLIAMS
Standard Slag Co.
PAUL M. WOODWORTH
Waylite Co.

Committee 214—Evaluation of Results of Compression Tests of Field Concrete

Organized 1946

The committee's assignment is to analyze data of compressive tests of samples of concrete taken in the field, to determine the nature and extent of variations in them for different job conditions, and to develop measures of the reliability of such tests as bases for the design of concrete proportions.

HUGH C. ROSS, Chairman
Hydro-Electric Power Com. of Ontario
ELWOOD H. BROWN
University of California
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Thompson and Lichtner Co., Inc.
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University of Maine
STANTON WALKER
National Sand and Gravel Assn.
CHARLES S. WHITNEY
Ammann and Whitney
CHARLES E. WUERPEL
Marquette Cement Manufacturing Co.

Committee 215—Fatigue of Concrete
Organized 1947

The assignment of this committee is threefold: (1) to review the available data on the behavior of plain and reinforced concrete under repeated loading, and to prepare a report on the present state of knowledge in the field; (2) to consider the implications of this knowledge in the design of members and structures of plain and reinforced concrete; and (3) to determine the gaps in our knowledge, and to recommend and encourage research which will provide the information to fill these gaps. A preliminary draft of a bibliography on fatigue of concrete was circulated in the United States and abroad to persons it was felt would be interested and who might contribute additions or corrections to list of references.

CHESTER P. SIESS, Chairman
University of Illinois
FRED BURGGRAF
Highway Research Board
HARMER E. DAVIS
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G. S. PAXSON
Oregon State Highway Dept.
CHARLES H. SCHOLER
Kansas State College
THOMAS G. TAYLOR
Concrete Specialties Co.
L. W. TELLER
Bureau of Public Roads
DAVID WATSTEIN
National Bureau of Standards

Committee 312—Plain and Reinforced Concrete Arches
Organized 1930

This committee has published three reports in the ACI JOURNAL on reinforced concrete arch design—correlating much that has been learned of the influence of dead and live loads and volume changes due to shrinkage, plastic flow and temperature change. The most recent report, May 1951 JOURNAL, presented a general design method, and a final report will be submitted as soon as an adequate standard can be prepared on current research.

CHARLES S. WHITNEY, Chairman
Ammann and Whitney
BOYD G. ANDERSON
Ammann and Whitney
W. S. COTTINGHAM
University of Wisconsin
CLYDE T. MORRIS
Ohio State University

Committee 314—Rigid Frame Bridges
Organized 1935

This committee is charged with accumulating data on the behavior of existing rigid frame bridges, with reference to the effects of temperature, placing schedules, dead and live loads, impact, etc. The committee is cooperating with the Institute of Transportation and Traffic Engineering, University of California, in a static and dynamic testing program of California highway bridges. The committee is planning a series of tests on type and scale of models and instrumentation with a view to having a final report ready by December 1953.

DAN H. PLETTA, Chairman
Virginia Polytechnic Institute
MILTON BRUMER
Ammann and Whitney
LEONARD C. HOLLISTER
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GEORGE J. KEREKES
Bechtel Corp.
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Iowa State College
ALFRED L. PARME
Portland Cement Assn.
FREDERICK A. REICKERT
University of Illinois
V. G. SZEBEHELY*
Virginia Polytechnic Institute

Committee 315—Detailing Reinforced Concrete Structures
Organized 1936

The committee has completed a revision of "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)" to bring methods of detailing into agreement with requirements of "Building Code Requirements for Reinforced Concrete (ACI 318-51)." The revision, undertaken with the cooperation of the Concrete Reinforcing Steel Institute, was approved at the 1951 convention and ratified by letter ballot April 5, 1951. The completely revised "Manual" was published March 1952. The committee is reviewing CRSI "Highway

*Non-Member ACI.

Structures Manual" for conformity with the ACI Manual and the possibility of combining them into a single volume. A new project is preparation of a teaching syllabus as aid in use of the "Manual."

RAYMOND C. REESE, Chairman
Consulting Engineer
RAYMOND ARCHIBALD*
Bureau of Public Roads
FRANK H. BEINHAUER
Consulting Engineer
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Bethlehem Steel Co.
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Office, Chief of Engineers, Dept. of the Army
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Veterans Administration
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Bureau of Reclamation
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Hammond Iron Works
ALLEN M. REESE
R. C. Reese, Consulting Engineer
E. E. RIPPSTEIN*
Laclede Steel Co.†
JOHN F. SEIFRIED
Ceco Steel Products Corp.

Committee 318—Standard Building Code

Organized 1929

The committee's original assignment was completed with the approval by ACI membership of "Building Code Requirements for Reinforced Concrete" (ACI 318-41). Since then the committee has twice revised the Code, in 1947 and 1951. It is currently continuing with the assignment of keeping the Code in line with current design and construction practice and correcting any provisions which do not prove satisfactory in actual use.

FRANK KEREKES, Chairman
Iowa State College
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Consulting Engineer
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Wire Reinforcement Institute
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A. BURTON COHEN
Consulting Engineer
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Portland Cement Assn.
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Los Angeles Dept. of Bldgs. and Safety
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Thomas, Jameson and Merrill
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†Corporation Member.

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Ohio State University
FRED F. MCMINN
Commissioner of Bldgs., Cincinnati
NOLAN D. MITCHELL
National Bureau of Standards
I. E. MORRIS
I. E. Morris Associates
NATHAN M. NEWMARK
University of Illinois
DOUGLAS E. PARSONS
National Bureau of Standards
RAYMOND C. REESE
Consulting Engineer
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O. J. Porter and Co.
A. CARL WEBER
Laclede Steel Co.
C. H. WESTCOTT
Westcott Engineering Co.
WALTER H. WHEELER
Consulting Engineer
C. A. WILLSON
American Iron and Steel Institute

Committee 321—Design of Reinforced Concrete Slabs

Organized 1941

This committee is assigned the problem of analysis and design of bridge and building slabs. The immediate objective will be the problem of floor slabs in buildings, including the effect of uniform and concentrated loadings. The committee is making a study of accurate solutions of the problem that have been developed in recent years, as well as of experimental data that have been made available; this with a view toward investigating the adequacy of current and proposed approximate methods of design. Plans are under way to run tests of different types of construction and design of flat and two-way slabs. Consideration is also being given to transmitting to the Reinforced Concrete Research Council material on multiple panels for concrete floors. It is hoped that the RCRC will be interested in supporting the program.

NATHAN M. NEWMARK, Chairman
University of Illinois
RENE L. BERTIN
Consulting Engineer
A. BURTON COHEN
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Portland Cement Assn.
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J. Di Stasio & Co.
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C. F. Braun and Co.
O. G. JULIAN
Jackson and Moreland
DAN H. PLETTA
Virginia Polytechnic Institute
CHESTER P. SIESS
University of Illinois
WALTER H. WHEELER
Consulting Engineer
C. A. WILLSON
American Iron and Steel Institute

Committee 323—Prestressed Reinforced Concrete—Joint ASCE-ACI
Organized 1942

This committee was assigned to review present knowledge of prestressed concrete, to develop design procedure and to recommend needed research. Reorganized as a Joint ASCE-ACI group in 1952, there are now five subcommittees whose activities have resulted recently in a bibliography of over 1000 references, study of patent development in U. S. and abroad, and a set of notations and list of definitions prepared with a view to submitting them to the Institute for publication in the JOURNAL.

A. E. CUMMINGS, Chairman
Raymond Concrete Pile Co.
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Bureau of Yards and Docks
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Bureau of Public Roads
P. G. BOWIE
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Preload Corp.
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American Steel & Wire Co.†
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Societe Technique pour l'Utilisation de la Precontrainte, Paris
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Illinois Institute of Technology
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Concrete Pipe Associations, Inc.
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Danish National Institute of Bldg. Research
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University of Illinois
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Massachusetts Institute of Technology
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Naval C. E. Research and Evaluation Lab.
CHARLES C. SUNDERLAND
John A. Roebling's Sons Co.
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The Austin Co.
C. C. ZOLLMAN
Vacuum Concrete, Inc.

Committee 324—Precast Reinforced Concrete Structures
Organized 1946

The committee reports completion of its preliminary assignment of a comprehensive survey of available information dealing with the design and construction techniques of precast concrete structures. Utilizing the compiled data of the survey, the committee is now engaged in the preparation of a standard applicable to precast concrete construction. The committee hopes to submit a final draft of "Proposed Minimum Requirements for Precast Concrete Construction" at the 1953 convention.

A. AMIRIKIAN, Chairman
Bureau of Yards and Docks
BOYD G. ANDERSON
Ammann and Whitney
K. P. BILLNER
Vacuum Concrete, Inc.
LOUIS P. CORBETTA
Corbetta Construction Co.
LEO H. CORNING
Portland Cement Assn.
A. E. CUMMINGS
Raymond Concrete Pile Co.
RALPH W. KLUGE
University of Florida
EZRA G. ODLEY
Bureau of Yards and Docks
CHARLES D. WAILES, JR.
C. D. Wailes Corp.

Committee 325—Structural Design of Concrete Pavements for Highways and Airports
Organized 1948

The committee's assignment is to develop a recommended practice for the design of structural details for concrete highway and airport pavement—slab dimensions, cross section, joints, reinforcement, etc.—on the basis of the best information available, and to keep such recommendations up to date as new information indicates need for revision. Active work is in progress.

L. W. TELLER, Chairman
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FRANKLIN B. BROWN
Wire Reinforcement Institute
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Granco Steel Products Co.
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National Crushed Stone Assn.
ROBERT HORONJEFF
University of California
J. D. LINDSAY
Illinois Division of Highways
P. E MASHETER*
Ohio Dept. of Highways
L. A. PALMER*
Bureau of Yards and Docks
G. S. PAXSON
Oregon State Highway Dept.
THOMAS B. PRINGLE*
Office, Chief of Engineers, Dept. of the Army
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C. A. WILLSON
American Iron and Steel Institute
KENNETH B. WOODS
Purdue University
F. N. WRAY
Highway Research Board

*Non-Member ACI.
†Corporation Member.

Committee 326—Shear and Diagonal Tension —Joint ASCE-ACI
Organized 1950

A Joint Committee of ASCE and ACI was formed in 1950 with the assignment of developing methods for designing reinforced concrete members to resist shear and diagonal tension consistent with the new ultimate strength design methods. The committee is also listed as a subcommittee of ASCE's Masonry and Reinforced Concrete Committee. An initial test program investigated shearing strength of a reinforced concrete slab under a centrally located concentrated load. A project is also under way at the University of Illinois, sponsored by Reinforced Concrete Research Council.

CHARLES S. WHITNEY, Chairman
Ammann and Whitney
C. A. WILLSON, Secretary
American Iron and Steel Institute
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Bureau of Public Roads
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AISI Fellow, National Bureau of Standards
ALFRED M. FREUDENTHAL* (ASCE)
Columbia University
EIVIND HOGNESTAD
University of Illinois
D. P. JENNY* (ASCE)
Portland Cement Assn.
R. L'HERMITE
Laboratoires du Batiment et des Travaux Publics
DOUGLAS MCHENRY
Portland Cement Assn.
SVEN T. A. ODMAN* (ASCE)
Swedish Cement and Concrete Institute
DOUGLAS E. PARSONS
National Bureau of Standards
RAYMOND C. REESE
Consulting Engineer
F. G. THOMAS* (ASCE)
Building Research Station, England

Committee 327—Ultimate Load Design —Joint ASCE-ACI
Organized 1952

A Joint Committee was formed in 1952 with the Subcommittee on Ultimate Load Design of the ASCE Committee on Masonry and Reinforced Concrete, Structural Division. The assignment is to evaluate and correlate theories and data bearing on ultimate load design procedures with a view to establishing them as accepted practice.

LEO H. CORNING, Chairman
Portland Cement Assn.
RAYMOND ARCHIBALD*
Bureau of Public Roads
EIVIND HOGNESTAD
University of Illinois
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C. F. Braun & Co.
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California Division of Highways
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Ohio State University
JOHN I. PARCEL
Sverdrup & Parcel
DOUGLAS E. PARSONS
National Bureau of Standards
RAYMOND C. REESE
Consulting Engineer
CHARLES S. WHITNEY
Ammann & Whitney

*Non-Member ACI.
†Corporation Member.

Committee 505—Design and Construction of Reinforced Concrete Chimneys
Organized 1922

Committee 505 was reactivated in 1948 for the purpose of studying "Proposed Standard Specifications for the Design and Construction of Reinforced Concrete Chimneys" to bring them to the point of a motion for adoption as an Institute standard. A final draft is anticipated for submission at the 1953 convention.

ERNEST A. DOCKSTADER, Chairman
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Thompson and Lichtner Co., Inc.
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Stone and Webster Engineering Corp.†
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Jackson and Moreland
EDMUND W. REEVE
Custodis Construction Co., Inc.
GEORGE SHERVINGTON
Consolidated Chimney Co.

Committee 604—Winter Concreting Methods
Organized 1929

This committee completed its original assignment when its report was published and adopted as ACI Standard "Recommended Practice for Winter Concreting Methods (ACI 604-48)." Under its new chairman, the committee is undertaking a review of the existing standard. It is believed the greatest need lies in direction of the small builder and revisions will be reviewed with this as primary objective.

M. F. MACNAUGHTON, Chairman
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Thompson and Lichtner Co., Inc.
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U. S. Waterways Experiment Station
LOUIS R. FORBRICH
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Canada Cement Co.
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Hydro-Electric Power Com. of Ontario
R. W. SPENCER
Southern California Edison Co.
LEWIS H. TUTHILL
Bureau of Reclamation
CHARLES E. WUERPEL
Marquette Cement Manufacturing Co.

Committee 609—Vibration of Concrete
Organized 1934

This committee has the assignment to revise and bring up to date a former report, "Recommendations for Placing Concrete by Vibration," ACI JOURNAL, Mar.-Apr. 1936, *Proc.* V. 32. As a preliminary step in its work the committee was canvassed for an expression of views with the object of developing those facts which would be generally accepted and to determine the points which require clarification. The results of this poll were summarized in the ACI JOURNAL, Jan. 1940, *Proc.* V. 36, p. 265.

The survey revealed problems for further study and the committee has been organized to encourage research to discover the basic effects within concrete, produced by vibrators of different design, and to determine how their characteristics can be used to the greatest advantage. The advent of air entrainment in concrete has made it necessary to give consideration to the removal of such air by vibration. Short papers for a possible symposium are planned with a view to later condensation as a revised standard.

HARMON S. MEISSNER, Chairman
Bureau of Reclamation
H. L. FLODIN*, Secretary
Portland Cement Assn.
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Thompson and Lichtner Co., Inc.
HARMER E. DAVIS
University of California
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National Bureau of Standards
J. KACMARIK
Chicago Pneumatic Tool Co.
ARTHUR A. LEVISON
Blaw-Knox Co.
C. W. PIERCE
Viber Co.
A. G. TIMMS
Bureau of Public Roads
LEWIS H. TUTHILL
Bureau of Reclamation
I. L. TYLER
Portland Cement Assn.
GEORGE W. WASHA
University of Wisconsin
MORTON S. WESTLUND*
Electric Tamper and Equipment Co.†

Committee 611—Inspection of Concrete
Organized 1935

The committee's 140-page *Manual of Concrete Inspection* was distributed to the membership July 1941, and is available (for $1.00) from ACI headquarters. The committee hopes to have a revision of the manual, including proper handling of test specimens on the job, overloading of truck mixers, and use of excessively wet consistencies, completed in 1952.

J. W. KELLY, Author-Chairman
University of California
DONALD C. ANDREWS
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Thompson and Lichtner Co., Inc.
DAVID EHRENPREIS
L. F. Davis and Co., Inc.
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Ammann and Whitney
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Bureau of Public Roads
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Corps of Engineers
T. C. POWERS
Portland Cement Assn.
WILFRID SCHNARR
Hydro-Electric Power Com. of Ontario
R. W. SPENCER
Southern California Edison Co.
LEWIS H. TUTHILL
Bureau of Reclamation
I. L. TYLER
Portland Cement Assn.
RODERICK B. YOUNG
Hydro-Electric Power Com. of Ontario

Committee 612—Recommended Practice for Curing Concrete
Organized 1936

This committee wrote a recommended practice for curing concrete, and is now considering Standards Committee recommendations witb a view to early availability of the report.

There are many scattered data on the subject of curing but little in the way of specific instructions as a guide to correct practice in curing. A study of these data has revealed the nature of the fundamental factors of curing concrete properly and has defined the range of optimum values for these factors. The committee is now sponsoring basic research at the National Bureau of Standards on curing methods. A symposium on curing was sponsored at the 1952 convention.

MARK MORRIS, Chairman
Iowa State Highway Com.
G. E. BURNETT
Bureau of Reclamation
ROY W. CARLSON
Consulting Engineer
H. F. CLEMMER
Government of the District of Columbia
W. R. JOHNSON
Corps of Engineers
F. V. REAGEL
Missouri State Highway Dept.
A. G. TIMMS
Bureau of Public Roads
PAUL M. WOODWORTH
Waylite Co.

Committee 613—Recommended Practice for Proportioning Concrete Mixes
Organized 1936

The committee's report (ACI JOURNAL, Nov. 1943, *Proc.* V. 40, p. 93) was adopted

*Non-Member ACI.
†Corporation Member.

as an ACI Standard at the 40th annual convention, 1944, and was ratified by letter ballot, becoming effective July 28, 1944, as ACI 613-44. The present report does not cover adequately the proportioning of mixes containing entrained air and the committee has been actively engaged in revising this report. A preliminary draft has been prepared.

WALTER H. PRICE, Chairman
Bureau of Reclamation
W. A. CORDON, Secretary
Bureau of Reclamation
CLAYTON L. DAVIS
Universal Atlas Cement Co.
A. T. GOLDBECK
National Crushed Stone Assn.
FRED HUBBARD
National Slag Assn.
FRANK H. JACKSON
Bureau of Public Roads
HENRY L. KENNEDY
Dewey and Almy Chemical Co.
THOMAS B. KENNEDY
U. S. Waterways Experiment Station
GUY H. LARSON
Wisconsin State Highway Com.
H. C. ROSS
Hydro-Electric Power Com. of Ontario
I. L. TYLER
Portland Cement Assn.
STANTON WALKER
National Ready Mixed Concrete Assn.

Committee 616—Recommended Practice for the Application of Paint to Concrete Surfaces

Organized 1936

This committee's report, "Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces" was adopted as an ACI Standard at the 45th annual convention and ratified by the membership by letter ballot June 29, 1949. The adopted standard was published in the September 1949 JOURNAL.

The committee is now studying paints, other than cement paints, for use on concrete surfaces. Development of a recommended practice for use of organic base paints is expected in 1952.

G. E. BURNETT, Chairman
Bureau of Reclamation
FREDERICK O. ANDEREGG
J. B. Pierce Foundation
T. E. CHISHOLM*
Hydro-Electric Power Com. of Ontario†
R. E. COPELAND
National Concrete Masonry Assn.
W. ELLIS GROBEN*
U. S. Forest Service
BERNARD A. MALM*
Sears, Roebuck and Co.
G. S. PAXSON
Oregon State Highway Dept.
CLARA SENTEL*
National Bureau of Standards

*Non-Member ACI.
†Corporation Member.

Committee 617—Specifications and Recommended Practice for Concrete Pavements and Bases

Organized 1937

A revision of "Specifications for Concrete Pavements and Bases (ACI 617-44," was adopted at the 47th annual convention, Feb. 20, 1951, and ratified by the membership by letter ballot Apr. 5, 1951. The revised Standard (ACI 617-51) was published in the May 1951 JOURNAL and subsequent discussion in Part 2, December 1951 JOURNAL.

H. F. CLEMMER, Chairman
Government of the District of Columbia
A. A. ANDERSON
Portland Cement Assn.
A. T. GOLDBECK
National Crushed Stone Assn.
W. E. HAWKINS*
North Carolina State Highway Dept.†
FRED HUBBARD
National Slag Assn.
FRANK H. JACKSON
Bureau of Public Roads
T. J. KAUER
Ohio Dept. of Highways
BAILEY TREMPER
California State Highway Dept.
STANTON WALKER
National Ready Mixed Concrete Assn.

Committee 620—Construction Joint Practice

Organized 1941

The task of this committee is to survey existing practice in preparation and treatment of construction joints, and to correlate available information with a view to preparing a statement of recommended practice. The committee has under consideration information relative to practice in mass concrete construction.

HARMER E. DAVIS, Chairman
University of California
A. L. PAULS*
J. A. Jones Construction Co.
R. W. SPENCER
Southern California Edison Co.
BYRAM W. STEELE
Consulting Engineer
LEWIS H. TUTHILL
Bureau of Reclamation
I. L. TYLER
Portland Cement Assn.

Committee 621—Aggregate Selection, Preparation and Handling

Organized 1942

The committee's field is comprehended in the title; its preliminary assignment is the correlation of available information in this field, looking toward practice recommendations.

A preliminary draft of an "Introduction" to the report has been completed and a

portion entitled "Selection" also completed by the chairman in accordance with recommendations of a subcommittee. Consolidation of reports is planned.

I. L. TYLER, Chairman
Portland Cement Assn.
A. T. GOLDBECK
National Crushed Stone Assn.
FRED HUBBARD
National Slag Assn.
W. R. JOHNSON
Corps of Engineers
BROR NORDBERG
Editor, *Rock Products*
R. W. SPENCER
Southern California Edison Co.
BYRAM W. STEELE
Consulting Engineer
A. G. TIMMS
Bureau of Public Roads
LEWIS H. TUTHILL
Bureau of Reclamation
STANTON WALKER
National Sand and Gravel Assn.

Committee 711—Precast Floor Systems for Houses

Organized 1934

Proposed "minimum requirements" for precast floor units (ACI JOURNAL, Jan. 1946), were approved at the 1946 convention and ratified by letter ballot July 1946 as ACI 711-46. An acceptable recommendation for minimum spacing of parallel reinforcement in precast joists is under consideration for inclusion in the proposed revision of "Minimum Standard Requirements."

F. N. MENEFEE, Chairman
University of Michigan
LEO M. LEGATSKI, Secretary
University of Michigan
RALPH W. ADAMS
Consulting Engineer
FRED C. BAMMAN
Maule Industries
S. J. CHAMBERLIN
Iowa State College
WARREN A. COOLIDGE
Dept. of Public Works, Nashville
R. E. COPELAND
National Concrete Masonry Assn.
LAURENCE G. FARRANT
Consulting Engineer
H. B. HEMB
Concrete Products, Inc.
C. F. MOLDE
Bouchard Construction Co.
GLENN MURPHY
Iowa State College
O. NEIL OLSON
Marquette University
GAYLE B. PRICE
Price Brothers Co.
J. W. WARREN*
Southern Cast Stone Co., Inc.
C. A. WILLSON
American Iron and Steel Institute

Committee 716—High Pressure Steam Curing

Organized 1941

Reactivated in 1950 because of an awakening of interest in high pressure steam curing, the new group began a study of the report of former Committee 716 (ACI JOURNAL, April 1944, *Proc.* V. 40, p. 409) considering possible additions to the old report including: (1) the requirement and method of testing pertaining to shrinkage and moisture movement of concrete products, (2) chemical reactions which occur during high pressure steam curing, and (3) preparation of adequate criteria for judging whether or not a product has received adequate curing for the intended use. A project is under way at the Research Foundation, Toledo, Ohio, on the physical properties of high-pressure steam cured blocks.

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Standing Committee for Simplified Practice Recommendation R163-48, Coarse Aggregates (Crushed Stone, Gravel and Slag). Commodity Standards Division, U. S. Dept. of Commerce

F. V. REAGEL
Missouri State Highway Dept.

Painter appointed ASTM treasurer

ASTM has announced the appointment of Robert J. Painter as treasurer. He will continue to be assistant secretary, to which office he was appointed in 1946. He also will continue as associate editor of the ASTM Bulletin.

ASTM technical committees meet

During ASTM Committee Week, March 3–7, a number of ASTM technical committees held sessions.

Specifications on portland, masonry and blended cements received considerable attention at the meeting of Committee C-1 on Cement. The committee will recommend adoption of a revision in the Specifications for Portland Cement (C 150) which will fully recognize the division between normal portland and air-entraining portland cements insofar as air content is concerned.

A proposed specification for sulfur mortars has been developed by Committee C-3 on Chemical-Resistant Mortars. This specification will cover sulfur mortars to be used in chemical-resistant construction and particularly for joining chemical-resistant masonry.

Freezing and thawing tests for evaluating aggregates and concrete will be recognized under ASTM designations as a result of action taken at the meeting of Committee C-9 on Concrete and Concrete Aggregates.

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DISCUSSION

Discussion closed January 1, 1952

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Earthquake Resistant Design Considerations—R. R. Martel
Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures—Charles S. Whitney, Boyd G. Anderson and Mario G. Salvadori
Multistory Buildings Designed to Resist Earthquakes—John J. Gould
Lightweight Concrete for Lower Construction Costs—J. A. Murlin
Designing for Continuity in Prestressed Concrete Structures—Alfred L. Parme and George H. Paris
Pumice—Lightweight Aggregate—Leslie I. Neher
Hydration Products Formed in Cement Pastes at 25 to 175 C—George L. Kalousek and Milton Adams

Discussion closed February 1, 1952

Oct. Jl. '51

Advances in Precast Floor Systems—F. N. Menefee
Manufacture and Use of Machine-Made Precast Structural Elements—A. G. Streblow
Tilt-up Construction in Western United States—F. Thomas Collins
Diagonal Tension in Reinforced Concrete Beams—Arthur P. Clark
Coral and Salt Water as Concrete Materials—John G. Dempsey
Load Carrying Capacity of Dowels at Transverse Pavement Joints—Henri Marcus

Discussion closed March 1, 1952

Nov. Jl. '51

Tilt-Up Construction Costs—F. Thomas Collins and Earl Bennetsen
Effect of Temperature and Surface Area of the Cement on Air Entrainment—E. W. Scripture, Jr., S. W. Benedict and F. J. Litwinowicz
Foundation for a Large Turbogenerator—Paul Rogers
Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars—R. M. Mains
Insulation for Protection of New Concrete in Winter—L. H. Tuthill, R. E. Glover, C. H. Spencer and W. B. Bierce

Introduction to Ultimate Load Design*

By LEO H. CORNING†

SYNOPSIS

Review of the history of ultimate load design. Cites precedent for ultimate load design here and in foreign countries with some consideration of the load factors they require.

HISTORY

Had this session been held 50 years ago, in all likelihood a larger percentage of the audience would have been familiar with the concepts to be discussed. The reason for this paradox is not difficult to find. Ultimate load design is basically a return to forgotten fundamentals.

At the turn of the century, with reinforced concrete in its infancy, engineers had only the observed behavior of test specimens to guide them in design. Of necessity the early pioneers in reinforced concrete had to be familiar with the actual behavior of the material. Therefore, many of the earlier design theories, on the strength of sections, such as those advanced by Ritter in 1899, Talbot in 1904, Withey in 1907 and Mensch in 1914, maintained good correlation between test results and calculated values. Since then, however, the determination of the strength of a section has drifted toward the acceptance of arbitrary rules virtually unrelated to actuality and this phase of our design procedure has become merely a numerical exercise.

FUNDAMENTAL STEPS IN STRUCTURAL DESIGN

To properly appreciate the role ultimate load design plays in design procedure it is important that the three fundamental steps by which any structure is designed be sharply singled out. These are the determination of the service loads the structure is to carry, determination of the forces, moments and deflections which these loads create, and the determination of the dimensions of the members to resist economically the moment and forces produced by some multiple of the service loads. Ultimate load design is concerned solely with the last step.

In contrast to prevailing design procedure in which internal stresses based on a linear stress-strain relationship are equated to allowable stresses, ultimate load design is based on equating the strength of a section to the stresses

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†Member American Concrete Institute, Manager, Structural and Railways Bureau, Portland Cement Assn., Chicago, Ill.

produced by ultimate loads. For the sake of clarity, the two basic differences are worth repeating: first, assumed elastic behavior versus established plastic behavior; second, working loads and allowable stresses versus ultimate loads and ultimate strengths.

It should be clearly understood that *ultimate load* as used in ultimate load design is some multiple of the *maximum* load to which the structure will be subjected. It is considerably in excess of the ordinary working or service loads and exceeds with an adequate margin of safety the maximum load on the structure that may be expected at rare intervals, or to which the structure may be subjected unexpectedly. It follows that although the ultimate strength is based on a recognition of plastic action near the ultimate load, at service loads the structure is well within the elastic range. It is in no way contemplated that structures will be designed for working loads with strains within the plastic range where permanent set would occur.

In the early days of reinforced concrete design, parabolic distribution of compressive forces was as well known as the straight line distribution is today, therefore many structures were designed on the basis of plastic action or, in other words, according to ultimate strength theory. Many of these structures are still standing without evidence of distress. For example, it is well known that one of the outstanding engineers of those days, L. J. Mensch, employed an ultimate strength theory in the design of most of his notable concrete structures. More recently, some of the large thin shell structures in this country have been designed in accordance with ultimate strength theories, not particularly for economy reasons but in order to be able to predict the actual factor of safety of these structures. The straight-line theory and use of allowable working stresses gave an erroneous and unsafe picture of the stability and moment-thrust ratio in the arches.

SPECIFICATIONS FOR ULTIMATE LOAD DESIGN

In recent years a few countries have permitted the use of ultimate load design procedure. In Russia it is mandatory that structures be designed by this method. Brazilian and Czechoslovakian building codes have for some years specified that ultimate load design procedure be used as an alternate method. The objective has been to overcome the deficiency of the straight line theory which greatly underestimates the compressive strength of a member. No figures are available on the over-all load factors or factors of safety recommended in the Czechoslovakian code but likely they are similar to the Russian requirements which specify that the ultimate load should be from 1.6 to 2.2 times the service loads, depending on the ratio of dead load to live load and the type of member. The Brazilian code specifies that the ultimate load should not be less than twice the service loads.

The reader will note that the term *over-all load factor* or factor of safety just referred to simply designates the ratio of the ultimate load to the sum of the design loads. In some of the papers which follow the term *load factor*

will also be used. This term designates a multiple of an individual design load.

PRACTICAL APPLICATIONS

Regarding practical application of ultimate load design, K. Hruban reported in the Dec. 1949 issue of *Concrete and Constructional Engineering*, " . . . since 1946 several large structures in Czechoslovakia have been designed by this method and the savings in steel without increasing the allowable working stresses has been about 10 to 20 percent." The late A. J. Boase, reporting on his survey of reinforced concrete practice in Brazil, stated "not only have the engineers of Brazil studied these methods (ultimate load design) but they have actually applied them. It should not be inferred that the ultimate design method has replaced the classic procedures there but good buildings have been built where this method was used . . . I visited one building with the designing engineer where this method had been employed with a resultant saving of about 15 percent." There is, therefore, ample precedent for the use of ultimate load theories of design.

CONCLUSIONS

In the papers to follow many reasons will be given why we should design by ultimate load theories. The fundamental concepts of this design method and research data on which these design concepts and theories are based will be explained and additional needed research will be discussed. The actual application of design procedures will be demonstrated and some thought-provoking ideas on the establishment of load factors advanced. It is hoped the discussions here will stimulate serious thought on ultimate load design and also that this theory will be put into practice to find out just what it can do. The author believes by its application a completely different and more realistic design concept will result. Under the prevailing straight line method, because allowable stresses are established by a code writing body, there is little need for the designer to consider the actual over-all load factor for the individual members of a structure or for the structure as a whole. Consequently, little thought is given by him to these factors and they may vary from comparatively low to extravagantly high values. Ultimate load design, by clearly setting forth the actual load factors for each type of load as well as the over-all load factors, emphasizes the primary purpose of structural design, that is, making structures carry service loads economically and safely.

Title No. 48-52

Why Design by the Ultimate Strength Theories?*

By BOYD G. ANDERSON†

SYNOPSIS

Discusses following reasons for ultimate load design: to bring the design of concrete members into one common rational basis; to make safety factor for all shapes the same; to rationalize the use of load factors so that different factors of safety can be assigned to different types of loading and different types of structures; to prevent uneconomical use of compressive reinforcement; to simplify design procedure; to better predict performance of structures subject to long-duration impulsive loads; and to determine capacities of prestressed concrete members.

INTRODUCTION

There is resistance to changes in engineering procedures just as there is resistance to any new forms. However, there are indications of dissatisfaction with present design methods which, quite justifiably, has been extended to the use of the straight line theory for proportioning reinforced concrete members.

Theoretical analysis of structures is improving constantly due to widespread interest in the methods of least work, slope deflection, relaxation methods and other derivatives and shortcuts. Possibly these theoretical analyses are accepted because of immediate usefulness or because they appeal directly to the vanity of the user, at any rate there has been little reluctance on the part of the engineering profession to take advantage of them. However, the tendency toward improving the methods of proportioning members has not been maintained in the same progressive manner even though the advantages may equal those obtained by the analysis and further, the data supporting them is more complete than that supporting the analysis.

STRAIGHT LINE DESIGN THEORY

While adherence to the straight line design method may stem from an instinctive dislike of empirical methods, it may also assume an excellence in this method which is not based on established facts. Earlier skepticism of the validity of straight line design is being confirmed by each increase in test data. As the shortcomings of the straight line method became increasingly evident, arbitrary adjustments are added to the design codes to

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†Member American Concrete Institute, Associate Partner, Ammann and Whitney, New York, N. Y.

account for these recognized discrepancies. The result is the inconsistency of designing members by an ultimate strength method when the load is axial, by a straight line method when flexure is added and by a mixture of methods when the member contains compression steel.

It appears from recent research on members subjected to combined bending and flexural stress that the most important part of the straight line theory retained by the codes is perhaps the most fallacious. These data indicate that the straight line method furnishes an erroneous indication of strength; the predicted and actual capacity often differing by as much as 100 percent, as much error as might be expected if continuous beams were analyzed as simple beams. The significance of this particular research is evident in as much as almost all members of every structure are subjected to forces that fall somewhere in the category of combined flexure and direct stress. Thus the straight line method does not produce either consistent or satisfactory results.

Recognizing the faults of the present design criteria, the question remains as to the best method of correcting this condition. One proposed corrective measure considers further modifications to the straight line theory. Such modifications would add an arbitrary correction for each concrete strength and a family of such corrections for varying steel strengths to the already belabored straight line theory. More important, the corrective measures still would be inaccurate as they would not consider an increase in bending capacity with increase in axial load.

ULTIMATE STRENGTH DESIGN THEORY

In contrast, ultimate strength methods give every indication of being both accurate and useful. The possibilities of this method may be summarized as follows with several facts pointing to immediate design advantages.

Universal application

An acceptable design method must provide all concrete members, whether subject to axial load, bending or combinations of bending and axial load, both with and without compression steel, with one common rational design procedure. Ultimate strength formulas satisfy this objective.

The results obtained for under-reinforced members without compression reinforcement under the action of bending, by the ultimate strength methods, are similar to the results obtained by the straight line theory. In these members the proportions depend on the effective lever arm of the steel. The arm predicted by ultimate strength methods is slightly greater for the more under-reinforced beams and slightly smaller for the more heavily reinforced beams. Though the difference in results is small the ultimate strength methods are in somewhat better agreement with test data. Because the lever arm is smaller, the heavily reinforced beams have a somewhat better safety factor for any given total load when designed by ultimate strength methods.

For axial loads without flexure the results would be similar to present results as the formulas would be empirically adjusted to agree with the ultimate strength formulas now specified by code.

For members subject to combined thrust and flexure, the ultimate strength method predicts the intimate relation of moment to thrust whereby the moment capacity increases up to a critical thrust. This behavior, verified by empirical data, provides a rational basis for designing arches, columns and frames of all kinds. It is interesting to note that in no case have the ultimate strength methods been disproved and in only a few cases have even minor changes, in recommended coefficients, been suggested by published test data.

Uniform safety factor

An acceptable method for designing members should provide a uniform safety factor throughout the structure. This means that the design will provide planned strength for each member and, where important, for the various parts of each member. While the straight line method provides fairly accurate results for certain individual structural components such as beams, the same accuracy is not provided in integrated structures, frames, and arches which have direct thrust as well as flexure acting in the members, or in Vierendeel and similar frames which have axial tension as well as bending moment acting on some members. Since ultimate strength methods predict capacity with reasonable accuracy, all members can be designed to conform to desired load factors. This consistent accuracy helps achieve the stress distribution predicted by the analysis without major strains and redistributions.

Overload factors

The design of sections by ultimate strength theories necessitates use of moments and thrusts which assure the safety of the structure. The only practical method of achieving a factor of safety in ultimate strength design is to increase the expected service loads by overload factors. The selection of overload factors can be simplified by comparison with the factors of safety used in present practice. For example, if an under-reinforced beam subject to flexure is designed by ultimate strength methods for basic overload factors of 2.0 for dead and live load, the capacity of the member will be practically identical to the capacity required by straight line design for the same loads. However, another loading combination must be considered in the design. As the moment capacity increases appreciably with increased thrust, up to certain critical moment-thrust ratios, it is not safe to exaggerate the loads causing large thrusts and small moments. Consequently the basic loading equation must be supplemented by an additional equation which will provide relatively small coefficients for loads causing *high thrusts* and *small moments* and larger coefficients for loads causing proportionately higher moments. As dead load is apt to provide less critical moment-thrust ratios, this specific design requirement and the improbability of large variations in dead load suggest the use of small dead load coefficients in the second loading combination.

The straight line design method of course does not need the second set of control factors because it is too inaccurate to predict the increase of member capacity with thrust which has been so well substantiated by test.

There are other practical advantages in designing for loading combinations which are greater than the service loads and many cases may be cited where consistency in design is improved by use of ultimate loading combinations. For example, in the design of members governed by elastic stability, the stresses caused by deflection increase more rapidly than increase in load and the member may fail under relatively small overload.

There are other similar cases in which a small change in live load without a similar change in dead load may cause failures because design by working stresses rather than ultimate strength formulas causes unrealistic assumptions and conclusions.

Compression reinforcement at yield point stress

Ultimate strength formulas anticipate compression reinforcement acting at yield point stress. This behavior has been verified by research and is partially recognized by the present codes. Recognition of this behavior, as well as correction of the present design criteria limiting maximum percentages of steel, permits better control over the member proportions and flexibility and provides means for reducing the forces caused by displacement and volumetric changes. Very often the strength required throughout a structure may be reduced by increasing the flexibility of a single member.

Simplified design approach

One of the most decided advantages of ultimate strength theories from the viewpoint of the designer is the simplified approach to the design of all members. Basically the design consists of determining the compressive area of concrete at ultimate load and determining from this the effective resisting lever arm. The computation can be made directly from the ultimate strength formulas or may be simplified by cut and try methods or graphs. Essentially one single stress pattern and one basic formula are applicable to all design conditions, resulting in simplified and easily visualized design procedures.

Dynamic loads

The ability of structures to resist impulsive type loads, particularly of the blast pressure variety, depends on the energy absorbed by the members up to the point of maximum permissible strain, as the absorption of energy occurs largely in motions exceeding the yield point of the steel except in over-reinforced members. Therefore, the load capacity depends completely on the ultimate strength and the strain through which this ultimate strength can be maintained. The same ultimate strength formulas as are proposed for the static design predict the yield point strength and the strength under continued strain, consequently these formulas predict structural behavior with reasonable accuracy and are directly applicable to the analysis and design of elements in dynamic motion.

Prestressed concrete design

The advantages of the ultimate strength methods in conventional design apply equally well to the field of prestressed concrete design. There is no reason to believe that the ultimate strength of prestressed members will be different from that of members reinforced with conventional steel. In fact with the proportions and steel determined by ultimate strength formulas and the degree of prestressing determined by strain conditions, there is evidence that the strength requirements for any load are appreciably less than required by straight line methods of computation.

SUMMARY

Ultimate strength methods can be used to design members for moments and thrusts obtained by conventional elastic analysis. The procedure is not to be confused with limit design, which considers the transfer of moment due to increased rates of strain at higher loads and for strains occurring largely after yield strength is reached at one or more points in the structure. Consequently, member proportions governed by elastic analysis are conservative because they are based on elastic strains while some plastic strains and consequently redistribution will occur at peak load.

While shear and bond limitations have not been clearly ascertained as yet, the same criteria presently used with the straight line method may be used equally well with ultimate strength design, the conditions of stress and strain in the member under service loads being essentially the same whether designed by straight line or ultimate strength methods. With the completion of the extensive bond and shear studies now underway better methods of evaluating and providing shear and bond strength may be determined, however the present status in no way eliminates the utility of the ultimate strength methods.

CONCLUSION

In conclusion it appears that every reinforced concrete design problem including the design of under-reinforced and over-reinforced beams with and without compression steel, and with any combination of direct stress and flexure can be treated by a single simple ultimate strength method with equal and in most cases much better over-all results than are obtained by straight line design methods. Furthermore, the same design methods may be applied directly to the design of prestressed structures and structures subject to dynamic loads, thus facilitating the analysis of these structures.

By providing the means for tougher and more resilient structures many clearance problems may be solved and important internal stresses reduced, thus permitting the use of concrete where it would otherwise be impractical. While there are certain refinements on minor points of the theory which are likely to cause discussion for some time to come, these have little effect on the suitability of the method and there is no reason why ultimate strength design methods should not be adopted for immediate use.

Discussion

By JOHN I. PARCEL*

INTRODUCTION

Mr. Anderson has presented a number of cogent reasons for the use of an *ultimate strength* method in the design of reinforced concrete members. It is assumed that by an ultimate strength theory of design is meant one in which the member is proportioned for some multiple of the service load which would give an ultimate load in excess of the maximum to which there is any reasonably likelihood of it being subjected, giving due consideration to the plastic action of the material in determining the carrying capacity of the member.

STRAIGHT LINE DESIGN THEORY

This is to be contrasted with the method commonly used where the member is proportioned for a set of working loads (*i.e.*, a loading that may be expected to occur frequently under normal conditions) at working stresses, which are taken low enough to provide a presumably adequate margin of safety to cover uncertainties such as defects in materials and workmanship and overload. No account is taken of the plastic behavior of the material nor is there any specific determination of the maximum load that can be carried.

This method, as is well known, ignores a large element of reserve strength affecting the ultimate carrying capacity of members in flexure; for example, the use of a working stress of $0.4\,f_c'$ gives a nominal safety factor of 2.5 while the actual factor, if the concrete strength is critical, may be in excess of 3.5.

WORKING STRESSES *vs* ULTIMATE CARRYING CAPACITY

Quite aside from this, it is believed that from the standpoint of the philosophy of design there is much to be said for the ultimate load method. A structure is essentially a machine for carrying loads and however much our conventional methods may mask it, the prime problem of the designer is to develop a structure which will carry safely and serviceably, the reasonable maximum loading which may occur, or at least the maximum for which we are willing to provide. Except as they affect the safe carrying capacity (safe must include consideration of excessive deflections and excessive cracking), *unit stresses* are of little significance. In a flexural member, except at low stresses, the unit stress in the concrete does not vary, even approximately, in proportion to the load, high local stresses may be without significance and the nominal stresses (computed by conventional methods) due to temperature, support settlement and such, may have little or no effect on the ultimate carrying capacity.

*Member American Concrete Institute, Vice-President, Sverdrup and Parcel Inc., St. Louis, Mo.

Our present specifications and designing methods, by focusing attention so prominently on *working stresses* rather than on ultimate carrying capacity, have had an unfortunate influence. It is doubtful that the condition of reinforced concrete design is as bad as in the case of structural steel, where for many years the value of 16,000 psi was held absolutely sacred, but even in the concrete field the undue emphasis on unit stress, as such, has clouded our thinking.

LOADING CONDITIONS

As Mr. Anderson has pointed out, in the case of structures having a wide variation in the live load to dead load ratio, no consistent factor of safety can be achieved by conventional methods; to do so we must either use some form of ultimate load theory or use different values for live and dead load unit stress.

Recognizing that the two basic uncertainties in design are the actual useful carrying capacity of the member and the extreme maximum load that may have to be carried, there seems to be a real advantage in dealing with the two items separately, rather than using a single blanket allowance to cover all uncertainties.

To a considerable extent defects in materials and workmanship can be controlled by intelligent design, specifications and rigid inspection. In structures with a long service life, the problem of dealing with an exceptional and quite indeterminate overload may become an economic question of how much we are willing to spend to provide for a possible but wholly improbable load condition.

While so-called limit design is not being discussed, it seems worth noting that there is a large group of reinforced concrete structures, statically indeterminate types such as continuous girders, frames and arches, which possess a great reserve strength due to the phenomenon of redistribution (the process by which an overstressed section unloads a part of its burden on its understressed neighbor). Advantage of this property cannot be taken and hence we cannot develop a really efficient and economical design except by applying an ultimate strength theory.

PRECEDENT FOR ULTIMATE LOAD THEORY

Long experience will endorse Mr. Anderson's suggestion that structural engineers are a conservative lot not easily converted to new and relatively untried methods, even when well supported by logic and experiment. Certainly there is nothing new in the concept of determining the strength of a member in bending on the basis of the plastic action of the material; it is at least as old as de Saint Venant's famous research on the strength of timber and cast-iron beams in the 1850's. Talbot's classic study embodied in *Bulletin* No. 4, University of Illinois Engineering Experiment Station, published nearly 50 years ago, contained complete formulas for the strength of concrete beams on the assumption of a parabolic variation of the stress-strain relation,

and this was reproduced in full in Turneaure and Maurer's first edition, one of the earliest treatises on reinforced concrete.

Dr. Hognestad in a recent bulletin lists some twenty different authorities who, during the past 30 years, have proposed variations of the general ultimate strength theory of reinforced concrete beams. In 1936 a full day's session of the International Association for Bridge and Structural Engineering was devoted to a discussion of the effect of plastic action in increasing the ultimate strength of both steel and concrete structures. In 1941 the American Society of Civil Engineers selected as the paper of the year and awarded its highest prize to Dr. van den Broek's paper on "Limit Design" in which it was advocated that elastic stresses be largely ignored and attention concentrated on the ultimate strength of the various structural elements.

These developments have affected our design methods very little. We are making much better concrete so we use higher stresses; otherwise, in most offices, we design just about as we did 40 years ago.

CONCLUSIONS

Mr. Anderson notes the appearance of a marked restlessness in the profession regarding our conventional design methods and it seems that of late there has been more interest shown, by various technical society committees, in the basic question of safety factors than ever before.

Our natural resources are not inexhaustible. The day must come when we can no longer say, "In case of doubt, throw in some more material." Many of us believe that the ultimate strength theory of design is an important aid in limiting the area of uncertainty and this, no doubt, explains the great increase of interest recently shown. Perhaps we should be patient and hopeful. There is a Latin proverb to which I presume we would all subscribe, *Magna est veritas et praevalebit*—"The truth is mighty and will prevail."

Title No. 48-53

Fundamental Concepts in Ultimate Load Design of Reinforced Concrete Members*

By EIVIND HOGNESTAD†

SYNOPSIS

The work of the pioneers of reinforced concrete design was directed primarily towards predicting the strength of members. Ultimate load design is, therefore, not a recent development. In principle it is older than the straight-line theory.

The fundamental concepts in ultimate load design of reinforced concrete members are discussed with particular emphasis on the basic assumptions involved, and most common design equations are presented.

It is pointed out that a desirable structure does not necessarily result from *truly balanced design*, which is characterized by several modes of failure being equally probable. The view is advanced that particularly undesirable modes of failure, such as those having a brittle nature, should be made less probable than modes of failure associated with ductility and warning of distress.

INTRODUCTION

Historical background

Ultimate load design is not a new development in strength of materials. A pronounced interest in the ultimate strength of structural members dates back only one or two decades, but its origin may be found far back in the records of engineering endeavor, further back than the concepts of elasticity and working stresses. The origin of systematic thought regarding flexure of beams, Galilei's work of 1638, was exclusively devoted to ultimate strength. Hooke's Law was formulated 40 years later, and over 180 years elapsed before the fundamental theorems of the theory of elasticity were developed by Navier in 1821.

Several early studies of reinforced concrete members, such as M. R. von Thullie's flexural theory of 1897 and W. Ritter's introduction of the parabolic distribution of concrete stresses in 1899, were ultimate load theories. The straight-line theory became generally accepted about 1900. Two primary reasons were given for such acceptance. The straight-line theory was mathematically simple, and the resulting safety factors, with respect to ultimate loads observed in tests, were sufficiently controlled to satisfy the requirements of that time.

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†Member American Concrete Institute, Research Assistant Professor, Theoretical and Applied Mechanics, University of Illinois, Urbana.

Ultimate load design is not new even in terms of American specifications. A committee report of the National Association of Cement Users suggested a reinforced concrete code in 1908 with the following design basis: "The design shall be based on the assumption of a load four times as great as the total working load (ordinary dead load plus ordinary live load), producing a stress in the steel equal to the elastic limit and a stress in the concrete equal to 2000 psi."

Such an ultimate design code was never adopted, however, since the first progress report of the first Joint Committee introduced the concepts of working loads and working stresses, thereby establishing the straight-line theory in 1909. Thus, conditions at working loads were emphasized in design, with relatively little attention being devoted to controlling the safety of designs.

Since then considerable progress has been made in reinforced concrete design and construction, and knowledge regarding the strength and behavior of reinforced concrete members has been greatly improved and extended. During the last two decades it has been suggested by concrete engineers all over the world that a return to load-carrying capacity as a primary consideration in design is possible and desirable. In American design a return to ultimate load design has been carried out for columns loaded concentrically or with small eccentricities, while the design of beams subject to flexure is still based on the straight-line theory. In other parts of the world, such as Brazil and in the Russian region of influence, a more general and consistent return to ultimate load design has been made.

This discussion of ultimate load design of reinforced concrete members is limited to fundamental concepts and the most common design problems. The available literature pertaining to the subject covered, especially foreign, is very extensive and therefore the material is presented without reference to the many original sources. Extensive bibliographies may be found in *Deutscher Ausschuss für Stahlbeton*, Heft 103, as well as in *Bulletin* 399 and *Circular* 64 of the University of Illinois Engineering Experiment Station.

Notation

The letter symbols used herein are defined below for convenient reference.

A_c = concrete gross area
A_{core} = area of column core (out-to-out of spiral)
A_s = area of tension reinforcement
A_s' = area of compression reinforcement
A_{st} = total area of longitudinal reinforcement
A_v = area of web reinforcement
a = distance from concentrated load to nearest support in a simple beam
α = angle between web bars and axis of beam
b = width of rectangular member
β = compression index

$$= \frac{A_s E_s \epsilon_u}{bd f_c'} = p \frac{E_s}{f_c'/\epsilon_u}$$

c = distance from neutral axis to compression edge of member
d = distance from centroid of tension reinforcement to compression edge of member
d' = distance between centroids of tension and compression reinforcements
E_c = modulus of elasticity of concrete
E_s = modulus of elasticity of reinforcing steel
e = eccentricity of load with respect to centroid of section
e' = eccentricity of load with respect to centroid of tension reinforcement
ϵ = strain

ϵ_u = ultimate concrete strain in flexure
ϵ_s = strain in reinforcement
f_c = compressive stress in concrete, also allowable stress
f_c' = compressive strength of 6 x 12-in. concrete cylinders
f_c'' = compressive strength of concrete in flexure = $k_3 f_c'$
f_s = stress in tension reinforcement
f_{sp} = useful limit stress of spiral reinforcement, generally assumed as the stress at a strain of 0.005
f_{yp} = yield point of reinforcement, especially tension reinforcement
f_{yp}' = yield point of compression reinforcement
K = $(\sin \alpha + \cos \alpha) \sin \alpha$
k_1, k_2 and k_3 = coefficients related to magnitude and position of internal compressive force in concrete
M = bending moment, also ultimate moment
n = E_s/E_c = modular ratio
o = circumference of reinforcing bar
P = load, also ultimate load
p = ratio A_s/bd ; p' = ratio A_s'/bd
p_{sp} = ratio of volume of spiral reinforcement to volume of concrete core
q = tension reinforcement index = $A_s f_{yp}/bd f_c' = p f_{yp}/f_c'$
q' = compression reinforcement index = $A_s' f_{yp}'/bd f_c' = p' f_{yp}'/f_c'$
r = ratio $A_c/(b\ s \sin \alpha)$
s = spacing of web bars along longitudinal axis of member
t = total depth of rectangular section
u = average nominal bond stress = $\dfrac{V}{jd\ \Sigma o}$
V = shearing force, also ultimate shear
v = nominal shearing stress in concrete = $\dfrac{V}{bj\,d}$

BASIC PHILOSOPHY OF ULTIMATE LOAD DESIGN

Fundamental steps in design

Ultimate load design may, as far as proportioning the various sections in a chosen structure is concerned, be outlined in four steps.

1. *Evalulation of Service Loads*—The dead loads are estimated from a preliminary design. The live loads are evaluated through a survey of the loads to be expected during the useful life of the structure. In common designs the live loads are given by building codes or similar documents.

2. *Evaluation of Ultimate Design Moments and Forces*—The dead loads are multiplied by one overload factor, the live loads by other, generally higher, overload factors. The magnitude of these overload factors depends on type of structure, type and probability of loads, exposure conditions and strictness of control in design and construction. Adding these multiplied service loads, using various combinations of live loads, gives the ultimate design loads, from which the ultimate design moments, shears, and axial forces acting in the various sections of the structure may be obtained. This determination of moments and forces from loads must, except for statically determinate members, be carried out with the aid of a theory of structures. Both the elastic theory and limit design have been propcsed for this purpose.

3. *Proportioning of Sections*—The sections of various structural members are proportioned with the aid of ultimate design equations involving forces and moments, sectional dimensions, and properties of the materials used.

4. *Performance under Service Loads*—The design of the structure must be checked for service loads to ensure that the structure will fulfill the function intended in design. This step involves such considerations as fatigue, creep, deflections, and width of flexural cracks in the concrete.

Design philosophy

Discussions in literature in this country and particularly abroad indicate that differences of opinion exist with respect to the detailed execution of

several of the foregoing outlined steps. This is especially so as far as the proportioning of sections is concerned, which is the major subject considered in this paper.

The primary source of disagreement regarding quantitative formulation of ultimate load design equations is the problem of judging to what extent the characteristics of the various modes of failure should be considered in design. The behavior of reinforced concrete members at high loads depends primarily on whether or not failure is initiated by general yielding of the reinforcing steel. Failures associated with yielding of reinforcement generally exhibit considerable ductility, while failures by crushing or fracture of the concrete have a brittle character.

A tension failure of a beam subject to bending, which is a typical example of failure initiated by yielding of the reinforcement, is characterized by considerable deformation taking place under increasing load after distress is indicated by considerable deflection and extensive cracking. Failures in shear of beams with web reinforcement and failures in compression of columns with spiral reinforcement possess similar characteristics. The ultimate strength of members failing in this manner is primarily dependent on the properties of the reinforcement, which are generally well known and well controlled in manufacture. Therefore uncertainty with respect to ultimate strength is relatively small.

Failures caused by overstressing of the concrete before yielding of the reinforcement generally are of a sudden and brittle nature. Compression failures of beams and tied columns, diagonal tension failures of beams without web reinforcement, and punching failures of slabs and footings are examples of failures of this type. The ultimate strength of these members is governed primarily by the properties of the concrete, which entail a considerably higher degree of uncertainty than those of the reinforcement.

The differences between the characteristics and consequences of these two groups of failures may have an important influence on the behavior of an overloaded structure. A multistory building with flat slab floors may well serve as a hypothetical example illustrating such an influence. If one of the upper floors is overloaded and fails initially in flexure by yielding of the reinforcement, ample warning is available in the form of large deflections and extensive cracking at a load when considerable flexural reserve strength is yet available. However, if the overloaded floor fails by punching around the columns, no warning is given since punching failures are sudden and violent in nature. Under these circumstances a local failure may lead to collapse of the floor, and the added load on the floor below may lead to progressive collapse of major parts of the building. The consequences of failure of a floor are accordingly very different. It is also important that the strength in bending of a flat slab floor be subject to a smaller degree of uncertainty than the punching strength.

On this dual basis, character of failure and degree of uncertainty with respect to the ultimate strength, one group of authors have supported the

view that certain modes of failure are more undesirable than others. They argue that reinforced concrete design should provide that the highly undesirable modes of failure be avoided. This may be achieved by rendering the overload factors, discussed previously, a function of the various modes of failure, or by purposely discounting the corresponding ultimate design equations used in the process of proportioning sections, that is, assuming for design purposes an *allowable* ultimate strength somewhat less than the actual one.

Another group of authors have asserted that the characteristics of the various modes of failure are irrelevant to design; structures are not supposed to fail; that is the purpose of overload factors. Accordingly, they have argued, the ideal *truly balanced* structure is one in which all modes of failure are equally probable.

Ultimate load design equations are based primarily on *facts*, that is on the properties of reinforced concrete observed in tests. Design philosophy as expressed by the two groups of *opinions* just outlined may also have a profound effect on the procedures of design. Therefore, both philosophies will be considered herein, in the discussion of design equations.

PROPERTIES OF THE MATERIALS

In the straight-line theory it is assumed that both concrete and reinforcement are elastic, the ratio between the moduli of elasticity being expressed by the modular ratio, n. For members subject to service loads of short duration, this is a reasonably good approximation. For estimating the ultimate strength of reinforced concrete members, the straight-line theory is not satisfactory and therefore, the properties of the component materials must be considered more realistically.

Load-time characteristics

The strength of reinforced concrete and its component materials depends to some extent on the time characteristics of the applied loads. Four types of loads are of primary importance in proportioning sections on an ultimate load basis: static short-time loads, sustained loads near the ultimate, repeated loads of magnitude near the ultimate, and impact loads. Most of our knowledge of the strength of reinforced concrete pertains to static short-time loading, and the discussion herein will be primarily concerned with such loads. In checking the performance of structures under service loads, sustained service loads and fatigue will also be of importance.

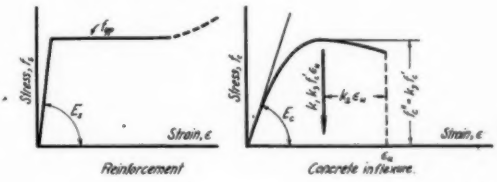


Fig. 1—Stress-strain relations

Reinforcing steel

The reinforcement normally used in this country has a flat yield point as indicated in Fig. 1. Hence, the reinforcement is actually an elastic-plastic rather than a purely elastic material. In most cases of practical design, failure of the concrete will take place before the steel reaches the strain-hardening range. The stress-strain relationship for the reinforcement, in tension as well as in compression, may therefore be assumed as a trapezoid with a yield level at the yield point stress, f_{yp}.

High strength reinforcement manufactured by cold twisting is used to some extent in Europe. Such reinforcement, which does not possess a flat yield point, is at present of limited importance in this country, and therefore, is not considered here.

Concrete

It was recognized by the pioneers of reinforced concrete design that concrete is an inelastic material with quite different properties in tension and compression. In this country the compressive strength of 6 x 12-in. cylinders, f_c', is generally used as a criterion of concrete strength. This strength standard has its deficiencies, but since no more satisfactory standard has been developed, cylinder strength will be used herein.

Compressive Strength—Compressive strength of concrete in structural members is primarily a function of the size and shape of the concrete mass considered and of the cylinder strength. For design purposes the compressive strength of concrete in beams, columns and other structural members may be conveniently expressed as 85 percent of the cylinder strength.

Tensile Strength—The tensile strength of concrete is only 7 to 10 percent of the cylinder strength, the percentage varying primarily with moisture content and compressive strength. In reinforced concrete members the tensile strength is of importance only insofar as shear and diagonal tension failures are concerned. Even though some tension must be carried between flexural cracks in reinforced concrete members subject to bending, this contribution to the strength of such members is small and unreliable.

Flexure—Bernoulli's Hypothesis asserting that a linear distribution of strains exists in flexure has been found to be reasonably true for reinforced concrete, even at loads near the ultimate strength. Accordingly, the distribution and magnitude of the compressive stresses in the concrete of a member subject to flexure depends primarily on the stress-strain relation of the concrete.

This stress-strain relation has been subject to considerable study in the past. It has been found that the relation is not necessarily the same in flexure and in compression. For instance, concrete in concentric compression will show signs of crushing shortly after the maximum stress is reached at strains of 1.5 to 2 thousandths, while crushing takes place at the compression face of members subject to flexure at strains of 3 to 5 thousandths. This beneficial effect of a stress gradient was observed by Talbot before 1910, and numer-

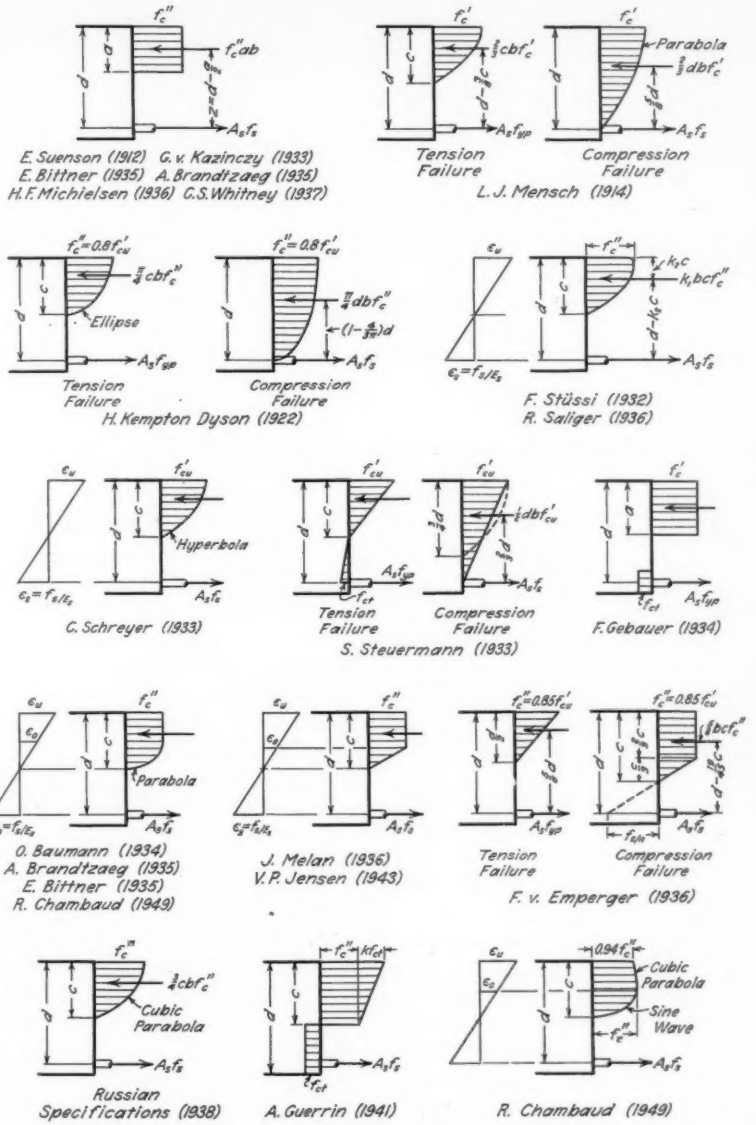


Fig. 2—Assumptions in flexural analysis (From University of Illinois Engineering Experiment Station, Bulletin 399)

ous later investigations have verified his findings. The exact form of the stress-strain relation of concrete in flexure is difficult to determine by direct experimental means since measurement of stresses is difficult although strains may easily be observed. A number of such relationships are shown in Fig. 2.

A general shape of the stress-strain relation for concrete in flexure is indicated in Fig. 1. For purposes of studying the behavior of reinforced concrete members from the smallest loads to failure, a mathematical relation between strain, ϵ, and stress, f_c, must be assumed. However, ultimate load equations may generally be developed, assuming only certain characteristics of the relation. The average stress may be expressed as follows (Fig. 1):

$$k_1 f_c'' = \frac{1}{\epsilon_u} \int_0^{\epsilon_u} f_c \, d\epsilon \quad \ldots\ldots (1)$$

The center of gravity is determined by

$$k_2 \epsilon_u = \frac{1}{k_1 \epsilon_u f_c''} \int_0^{\epsilon_u} f_c (\epsilon_u - \epsilon) \, d\epsilon \quad \ldots\ldots (2)$$

The maximum stress in flexure, f_c'', is related to the cylinder strength, f_c', by:

$$k_3 f_c' = f_c'' \quad \ldots\ldots (3)$$

Finally, the ultimate strain in flexure, ϵ_u, must be assumed. Recent tests in this country and abroad have indicated that ϵ_u is about 3 to 5 thousandths if a member is loaded to failure in a few hours.

The constants k_1, k_2 and k_3, whose use was introduced by F. Stüssi in 1932, are functions primarily of concrete strength. Values of k_1 from 0.70 to 0.90, of k_2 from 0.35 to 0.45, and of k_3 from 0.85 to 1.0 have generally been suggested.

Bond between concrete and reinforcement

Interaction between concrete and reinforcement depends on the bond between the two. This bond must prevent a general slip in regions of high shear. The bond strength is furthermore a factor governing the spacing of flexural cracks in the concrete since local high bond stresses must exist at such cracks even in regions without shear. Deformed bars meeting ASTM designation A 305-50T develop such high bond stresses that it is difficult to produce general bond failures in beams. Ultimate average nominal bond stresses greater than 0.20 f_c' have commonly been found. It is therefore the primary function of deformed reinforcement to limit the width of flexural cracks by limiting their spacing, and it may be assumed in developing ultimate load equations that no general slip occurs between reinforcement and concrete.

ULTIMATE LOAD DESIGN EQUATIONS

Ultimate load design equations may be developed on the basis of the assumed properties of the materials with the aid of equations of equilibrium and compatibility. The most important types of loading are discussed below.

Flexure

With reference to Fig. 3 the following equations are available:

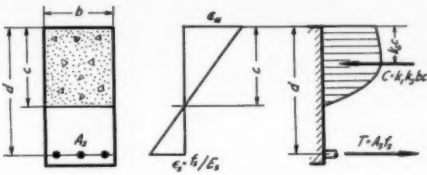


Fig. 3—Flexural analysis

$$A_s f_s = k_1 k_3 bc f_c' \quad (4)$$

$$M = k_1 k_3 bc f_c' (d - k_2 c) \quad (5)$$

$$\epsilon_s = \epsilon_u \frac{d - c}{c} \quad (6)$$

Tension Failure—A tension failure is initiated by yielding of the tension reinforcement with large deformations taking place before crushing of the concrete at the compression face. At failure the stress in the tension reinforcement, f_s, is accordingly at the yield point, f_{sp}, and an equation for the ultimate load of such beams may be developed without the aid of compatibility, Eq. 6. Solving Eq. 4 and 5 we obtain:

$$\frac{M}{bd^2 f_c'} = q\left(1 - \frac{k_2}{k_1 k_3} q\right) \quad (7)$$

in which the tension reinforcement index,

$$q = \frac{A_s f_{sp}}{bd f_c'} = p \frac{f_{sp}}{f_c'}$$

Eq. 7 includes the majority of the expressions which have been suggested for the ultimate strength of beams failing in tension. For instance, R. Saliger, V. P. Jensen, K. C. Cox and others have suggested $k_2/k_1 = 0.5$ and $k_3 = 1.0$. C. S. Whitney used $k_2/k_1 = 0.5$ and $k_3 = 0.85$, and the writer has found $k_2/k_1 = 0.55$, $k_3 = 0.85$. These differences in the values of k_1, k_2 and k_3, which characterize the stress block in the concrete at failure, are actually of minor importance in design. The greatest difference that may reasonably be considered is shown in Fig. 4. It appears that the difference in ultimate moment as given by Eq. 7 is less than 10 percent, which is small compared to other uncertainties involved in reinforced concrete design.

A tension failure of reinforced concrete beams is in several respects less undesirable than any other mode of failure. Large deflections give warning of approaching failure, the ability to absorb energy from impact and blast is relatively high and failure is unlikely to result in total collapse. The ultimate moment depends primarily on the yield point of the steel, the concrete strength being of minor importance. Hence, the ultimate moment is little affected by possible local inferiorities of the concrete.

Both schools of thought referred to previously have suggested the use of Eq. 7 as an ultimate load design equation. To determine the proper value for the ratio k_2/k_1k_3, a statistical study of all reliable test results available, well over 1000 beam tests, should probably be made. For practical purposes, however, any value of k_2/k_1k_3 from $\frac{1}{2}$ to $\frac{2}{3}$ seems acceptable.

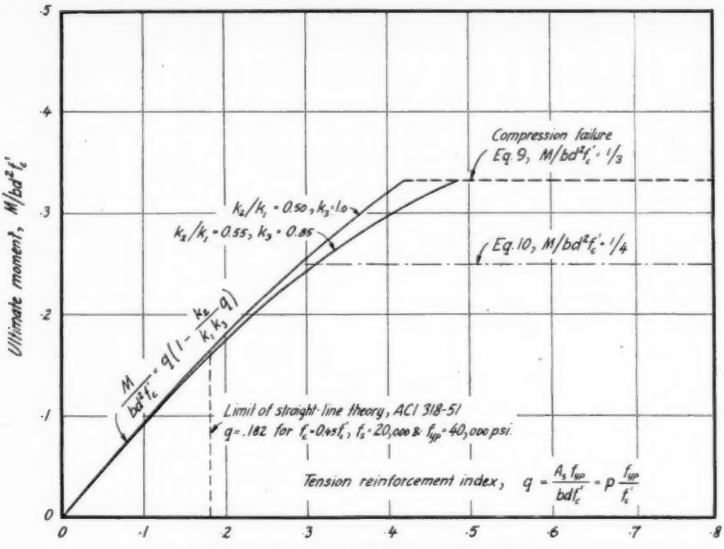


Fig. 4—Ultimate strength of beams failing in tension

Compared with the straight-line theory, Eq. 7 has several agreeable features, most important of which is mathematical simplicity. Furthermore, the effect of the quality of the reinforcement is recognized in a realistic manner so that possible advantages resulting from the use of high strength reinforcement may be realized.

Compression Failure—A compression failure takes place by crushing of the concrete at the compression edge with stresses in the tension reinforcement below the yield point. The compatibility Eq. 6 must therefore be used to determine the steel stress, f_s, at failure. Solving Eq. 4, 5 and 6 we find

$$\frac{M}{bd^2 f_c'} = \beta\left[\tfrac{1}{2}\left(1 + \frac{k_2}{k_1 k_3}\beta\right)\left(\sqrt{1 + \frac{4k_1 k_3}{\beta}} - 1\right) - k_2\right] \quad \ldots\ldots\ldots\ldots (8)$$

in which the compression index,

$$\beta = \frac{A_s E_s \epsilon_u}{bd f_c'} = p\frac{E_s}{f_c'/\epsilon_u}$$

The index β in this equation is similar in nature to pn, used in the straight-line theory. The three factors k_1, k_2 and k_3, which characterize the distribution and magnitude of the compressive stresses in the concrete at failure, have been given various values. Reasonable values for k_1 are from 0.7 to 0.9, for k_2 0.35 to 0.45, the values increasing with decfeasing concrete strength. The factor k_3 has been given values from 0.85 to 1.0.

Eq. 8 is plotted in Fig. 5 with two sets of values for k_1, k_2 and k_3. It appears that the ultimate moments resulting from the higher values of k_1 and k_3

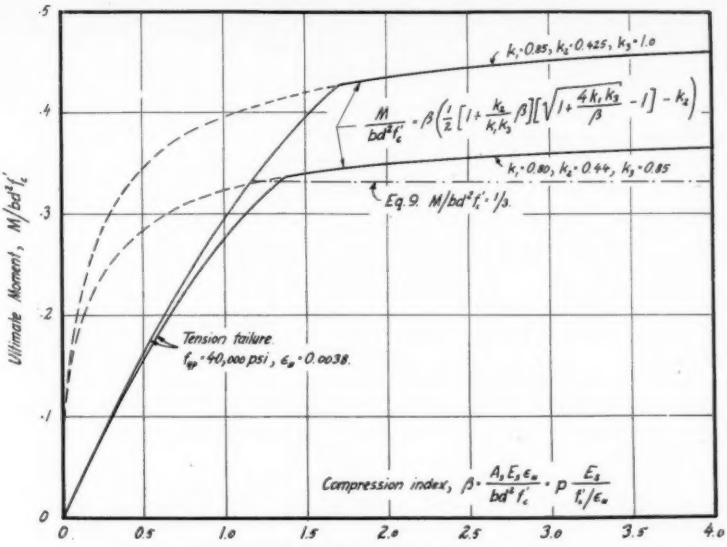


Fig. 5—Ultimate strength of beams failing in compression

are about 25 percent over the lower curve. Test results indicate that the ultimate moments of beams with concrete strengths as low as 1000 psi are near the upper curve while concrete strengths from about 3000 to 6000 psi result in values near the lower curve.

Fig. 5 further indicates that for normal reinforcement strengths, the ultimate moments for compression failures are relatively insensitive to the index β and accordingly to the reinforcement percentage, p, and the ultimate strain in the concrete, ϵ_u. This is not necessarily so for members prestressed with high strength wires or cables. The equation developed by C. S. Whitney has considerable merit for concrete strength over 3000 psi:

$$\frac{M}{bd^2 f_c'} = \tfrac{1}{3} \quad \text{.....(9)}$$

This equation is conservative for low concrete strengths. However, its outstanding simplicity is indeed agreeable, and conservatism may wisely be exercised with respect to low concrete strengths which are generally associated with considerable strength variations.

Eq. 8 and 9 predict the strength of beams failing in compression, a failure characterized by brittleness and an abrupt reduction in load-carrying capacity. Uncertainties with respect to concrete strength are reflected in the ultimate moments almost to the extent of proportionality, and relatively large creep deformations may develop at service loads.

As indicated previously it has therefore been suggested that discounted equations should be used in ultimate load design; for instance, a discount of

25 percent gives:

$$\frac{M}{bd^2 f_c'} = \tfrac{1}{4} \quad \text{(10)}$$

In Fig. 4 it is seen that the use of Eq. 10 rather than Eq. 9 is the equivalent of reducing the highest tension reinforcement index, q, permitted without compression reinforcement from about 0.45 to 0.30. An even stricter limitation is used in the straight-line theory of the ACI Building Code 318-51. For *balanced* design with $f_c = 0.45\, f_c'$, $f_s = 20{,}000$ psi, and $n = 30{,}000/f_c'$ we find $q = 0.453\, f_{yp} \times 10^{-5}$, or for $f_{yp} = 40{,}000$ psi, $q = 0.182$.

For compression failures of beams with compression reinforcement, C. S. Whitney modified Eq. 9 as follows:

$$\frac{M}{bd^2 f_c'} = \tfrac{1}{3} + q' \frac{d'}{d} \quad \text{(11)}$$

in which the compression reinforcement index, $q' = A_s' f_{yp}'/f_c'$ and d' is the distance between the centroids of compression and tension reinforcement.

The corresponding deducted equation may be written:

$$\frac{M}{bd^2 f_c'} = \tfrac{1}{4} + q' \frac{d'}{d} \quad \text{(12)}$$

Compared to the straight-line theory, Eq. 9 to 12 not only reflect the actual inelastic properties of steel and concrete in a more realistic manner, but are much simpler. The validity of the ultimate Eq. 7, 9, and 11 is well established by numerous tests.

Axial compression

About 1900, concentrically loaded reinforced concrete columns were generally designed by the following equation for the allowable load:

$$P = f_c A_c + f_s A_{st} \quad \text{(13)}$$

in which f_c and f_s are allowable stresses and A_c and A_{st} are areas of concrete and longitudinal reinforcement, respectively. This early equation recognized the inelastic properties of the component materials, which is by no means a coincidence. At the turn of the century A. Considère indicated that the ultimate strength of tied columns "varies little from the sum of the resistances offered by the crushing strength of the concrete and the longitudinal bars when stressed up to their elastic limit," and furthermore "The compressive resistance of a hooped member exceeds the sum of the following three elements: (1) compressive resistance of the concrete, (2) compressive resistance of the longitudinal rods stressed to their elastic limit, and (3) compressive resistance which would have been produced by the imaginary longitudinals at the elastic limit of the hooping metal, the volume of the imaginary longitudinals being taken as 2.4 times that of the hooping."

Indeed, the early designers were well familiar with the principles governing the ultimate strength of columns. However, when the straight-line theory was established, the transformed area equation came into use. This equation was used over three decades until ultimate load equations were re-established through the ACI column investigation in the 1930's. Then, the ultimate load

of tied columns and the yield point of spiral columns were expressed as

$$P = 0.85 f_c' (A_c - A_{st}) + f_{yp} A_{st} \quad \ldots\ldots (14)$$

and the ultimate load of spiral columns was given as

$$P = 0.85 f_c' (A_{core} - A_{st}) + f_{yp} A_{st} + 2.0 p_{sp} A_{core} f_{sp} \quad \ldots\ldots (15)$$

Following the publication of these experimental results, ultimate strength rather than working stresses was introduced as a basis for design of concentrically loaded columns in this country as well as in several countries abroad. The quantitative formulation of the design equations is an example illustrating that opinion as well as the facts represented by Eq. 14 and 15 may influence design. A minority of the ACI committee which carried out the column investigation recommended that an equation of the same form as Eq. 15 be used in design with a safety factor of 3.5:

$$P = \frac{1}{3.5}\left[0.85 f_c' (A_c - A_{st}) + f_{yp} A_{st} + 2.0 p_{sp}' A_{core} f_{sp}\right] \quad \ldots\ldots (16)$$

in which p_{sp}' is the effective percentage of spiral reinforcement over that necessary to *replace* the concrete shell:

$$p_{sp}' = p_{sp} - \frac{0.85 f_c'}{2.0 f_{sp}}\left(1 - \frac{A_c}{A_{core}}\right)$$

For tied columns $p_{sp}' = 0$ and Eq. 16 reduces to Eq. 14 with a safety factor of 3.5. This type of equation is commonly used in Europe. The majority of the ACI committee, however, recommended that design be based on Eq. 14, and the equations used in the present ACI Building Code 318-51 were later developed on the basis of such a view:

For spiral columns $P = 0.225 f_c' A_c + 0.40 f_{yp} A_{st}$ ..... (17)

For tied columns $P = 0.180 f_c' A_c + 0.32 f_{yp} A_{st}$ ..... (18)

Eq. 16 through 18, though reduced to fit the framework of building codes otherwise based primarily on the straight-line theory, are actually ultimate load equations derived from Eq. 14 and 15.

Eq. 16 is based primarily on the facts represented by Eq. 15. The safety factor of 3.5, however, is larger than the factors for tension failures of beams resulting from the straight-line theory, which in ACI 318-51 are about 1.8, 2.0, and 2.5 for structural, intermediate and hard grade steel, respectively. This difference represents an influence of opinion on judgment.

Eq. 17 and 18 represent an even more extensive influence of design philosophy. First, the design of spiral columns, Eq. 17, are based on the yield point load given by Eq. 14, not on the ultimate load given by Eq. 15, which may be developed only after spalling of the concrete shell. The increased toughness and the reserve strength of spiral columns is recognized, however, by the use of a lower factor of safety in Eq. 17 than in Eq. 18. Secondly, in deriving Eq. 17 and 18 from Eq. 14, a 50 percent higher safety factor has been used for the concrete than for the steel term. This was done in an effort to recognize the tendency for high steel stresses due to time yield and shrinkage. The 50 percent difference may also be interpreted as recognizing that a higher degree of uncertainty is associated with concrete strength, f_c',

than with the yield point of the reinforcement, f_{yp}. Finally, Eq. 17 and 18 as compared to Eq. 14 indicate factors of safety of the order of 3 to 4 as compared to about 2 for beams failing in tension. This may be interpreted as indicating that those preparing ACI 318-51 desired to recognize compression failures of columns as more undesirable than tension failures in beams by the use of 50 to 100 percent larger safety factors.

In establishing ultimate load design equations these matters of design philosophy may be considered in addition to the experimental facts. It must first be decided whether the design of spiral columns should be based on the yield point load, Eq. 14, or the ultimate strength, Eq. 15. The writer believes that since the strengthening effect of the spiral can only be realized after very large longitudinal deflections and spalling of the concrete shell, and since tests have indicated that even small eccentricities of load will lead to such large lateral deflections that no additional strength is developed after loss of the shell, the design of both tied and spirally reinforced columns should be based on the yield point load, Eq. 14.

The advocates of *truly balanced design* may wish to use Eq. 14 directly as an ultimate load equation. If, on the other hand, it is recognized that compression failures of columns are more undesirable than tension failures of beams, that spiral columns are tougher than tie columns, that it should be attempted to reduce the effects of creep, and that a higher degree of uncertainty is associated with f_c' than with f_{yp}, then the following type of discounted equations may be proper.

For spiral columns: $$P = 0.60\, f_c'\, A_c + f_{yp}\, A_{st} \quad (19)$$

For tied columns: $$P = 0.48\, f_c'\, A_c + 0.8\, f_{yp}\, A_{st} \quad (20)$$

The concrete term in Eq. 19 is discounted by 30 percent, somewhat more than the 25 percent discount suggested for compression failures of beams in Eq. 10. The discounted strength of a tied column, Eq. 20, is taken as 80 percent of the spiral column, which is the value used in ACI 318-51.

Combined flexure and axial compression

Ultimate load equations for reinforced concrete members subjected to combined flexure and axial compression were developed by A. Brandtzaeg about 1935. His study of tension failures may be included as a special case of that given below.

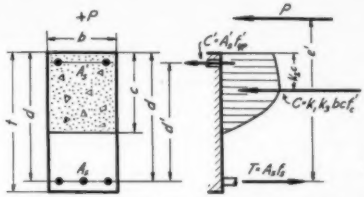
Tension Failure—A rectangular section with reinforcement at opposite faces is considered loaded in the plane of symmetry as indicated in Fig. 6. Since the ultimate strain in the concrete is 3 to 5 thousandths while the yield point strain of the reinforcement generally is 1 to 2 thousandths, both tension and compression reinforcement may be assumed to be strained over the yield point at failure. Using the notation of Fig. 6 and taking moments about the tension steel

$$Pe' = k_1\, k_3\, f_c'\, bc\, (d - k_2\, c) + A_s'\, d'\, f_{yp}' \quad (21)$$

Equilibrium of forces gives

$$P = k_1\, k_3\, f_c'\, bc + A_s'\, f_{yp}' - A_s\, f_{yp} \quad (22)$$

Fig. 6—Combined flexure and axial load



As for tension failures of beams, no compatability equation is needed. Solution of Eq. 21 and 22 gives

$$\frac{P}{bd\,f_c'} = q' - q + \frac{k_2}{k_1 k_3}\left[-\left(\frac{e'}{d} - 1\right) + \sqrt{\left(\frac{e'}{d} - 1\right)^2 + \frac{4k_2}{k_1 k_3}\left\{q'\frac{d'}{d} + \frac{e'}{d}\,(q - q')\right\}}\right] \quad \ldots\ldots (23)$$

in which the indexes are $q' = \dfrac{A_s' f_{yp}'}{bd\,f_c'}$ and $q = \dfrac{A_s f_{yp}}{bd\,f_c'}$

Similarly to Eq. 7, Eq. 23 includes the majority of the suggested expressions, and the effect of differences in the suggested values of k_1, k_2 and k_3 is relatively small, any value of $k_2/k_1\,k_3$ from $\frac{1}{2}$ to $\frac{2}{3}$ being acceptable in ultimate load design.

A tension failure of members subject to combined flexure and axial compression has similar desirable characteristics as tension failures of beams. Both schools of thought have therefore suggested Eq. 23 as an ultimate load design equation without any discount.

Compared to the straight-line theory, which involves the solution of a cubic equation, Eq. 23 represents a considerable mathematical simplification.

Compression Failure—While the design for tension failures given by ACI 318-51 is based on the straight-line theory, the design of columns with small eccentricities is based on ultimate strength. This was achieved by modifying Eq. 17 and 18 so as to provide a transition between axial compression and pure flexure.

Ultimate load equations for compression failures of members subject to combined flexure and axial load have been developed in the literature on the basis of the properties of concrete and steel discussed previously. This may be done in a manner similar to that indicated for beams, Eq. 8. The combined stress problem is mathematically more complex, however, since the solution of a cubic equation is necessary. Though somewhat simpler than the straight-line theory, such procedures hardly seem suitable for design except perhaps for complex sections. This is particularly so since simple approximate procedures have been presented for the most common cases without any serious loss of accuracy.

C. S. Whitney developed the following equation on a semi-empirical basis

$$P = \frac{2\,A_s' f_{yp}}{\dfrac{2e}{d'} + 1} + \frac{bt\,f_c'}{\dfrac{3\,te}{d^2} + 1.18} \quad \ldots\ldots (24)$$

This equation represents a transition between axial compression and pure flexure which has been found to be in satisfactory agreement with test results. Eq. 24 reduces to Eq. 11 for $e = \infty$. It reduces to Eq. 14 for $e = 0$ only if $2\ A_s' = A_{st}$ or, in other words, if the reinforcement is symmetrical, $A_s' = A_s$. Even if these conditions are not strictly met, it has been found that Eq. 24 gives a reasonably good approximation.

Compared with the straight-line theory and the more rational ultimate load theory, both of which involve the solution of a cubic equation, Eq. 24 is again mathematically simple.

Eq. 24 should be used as an ultimate load design equation if *truly balanced design* is desired. The corresponding discounted equation may be found as a transition between Eq. 12 and 20

$$P = \frac{2\ A_s'\ f_{yp}}{\frac{2e}{d'} + 1.25} + \frac{bt\ f_c'}{\frac{4te}{d^2} + 2.08} \qquad \text{.....(25)}$$

This equation reduces to Eq. 12 for $e = \infty$ and to Eq. 20 for $e = 0$.

Other Sections and Types of Loading—Ultimate load equations similar to Eq. 23 and 24 have been developed by C. S. Whitney for round columns. F. Gebauer, R. Saliger and others have developed expressions for the ultimate load of T-beams subject to eccentric axial loads and of rectangular sections subject to combined unsymmetrical flexure and axial load. Ultimate load theories have also been advanced which are sufficiently general to permit analysis of an arbitrary section. Some knowledge also exists regarding the ultimate strength of slender columns.

Special Considerations in Design—Eccentrically loaded members will deflect laterally under load, and such deflections may in some cases tend to increase the eccentricity of loading, thereby reducing the load-carrying capacity. For slender columns having an unsupported length greater than ten times the least lateral dimension, lateral deflections may have an important bearing on the ultimate strength and must be accounted for in a long column formula. For short columns, however, it seems sufficiently accurate simply to increase the eccentricities, e or e', by 5 percent.

Reinforced concrete members should generally be proportioned with the aid of ultimate load design equations on the basis of moments and forces multiplied by proper load factors. This is also the case for members subject to combined flexure and axial loads. In some structural applications, however, an overload may lead to increases in moment without proportional increases in axial load, and such a loading may govern the necessary section of the member. Particularly for members with large eccentricities, Eq. 23, the design therefore should be made not only for multiplied axial forces combined with multiplied moments, but also for direct *service* values of axial force combined with moment multiplied by proper load factors.

Shear and diagonal tension

In 1907–08, A. N. Talbot demonstrated that strength of reinforced concrete

beams failing in shear depends on a number of variables such as concrete strength, percentage of tension reinforcement, ratio of depth to span of beams as well as the amount, type and quality of web reinforcement. Talbot presented experimental evidence which qualitatively indicated effects of these variables.

About the same time, E. Mörsch derived expressions for the strength contribution of web reinforcement and developed the well-known expression for the nominal shearing stress, v.

$$v = \frac{V}{bjd} \qquad (26)$$

Considerable experimental and analytical study has since been devoted to reinforced concrete subject to shear. Qualitatively, knowledge of the behavior of beams failing in shear has been advanced very little, though some quantitative effects of the variables defined by Talbot have been evaluated. One of the first expressions for the ultimate strength of beams failing in shear was presented by O. Moretto in 1945.

$$v = \frac{V}{bjd} = Kr f_{vp} + 0.10 f_c' + 5000 p \qquad (27)$$

This equation was based on tests of simple beams loaded at the third-points.

Another ultimate load equation, including the effect of method of loading was developed by A. P. Clark in 1951

$$v = \frac{V}{bjd} = 2500 \sqrt{r'} + 0.12 f_c' \frac{d}{a} + 7000 p \qquad (28)$$

This equation was not recommended for general application as the web reinforcement used in the tests from which it was derived consisted exclusively of ⅜-in. vertical stirrups with a yield point of about 48,000 psi.

Comparison of Eq. 27 and 28 indicates that our knowledge of the ultimate strength of simple beams failing in shear is indeed limited. The effect of the percentage of web reinforcement, r, for instance, is given a considerably different effect in the two expressions. We know little about the strength of simple beams failing in shear. The strength of continuous beams, of members subject to combined flexure, axial load and shear, and of members subject to combined flexure, shear and torsion, is as yet hardly touched by experimental investigation. In the latter three cases we do not know how the members act when failure is approached. Shear in reinforced concrete members is a fruitful field for future investigation.

Since the ultimate strength is not known with accuracy for an arbitrary case met in design, *truly balanced design* of reinforced concrete members with respect to shear failures is, as yet, not possible. If, on the other hand, the point of view is applied to design that shear failures are brittle and sudden in nature and are accordingly to be avoided, then a reasonably reliable ultimate load design is possible. Two major cases arise in such design. First, under which circumstances may members safely be designed without any web reinforcement? Second, how much web reinforcement is necessary in

cases of heavier shear to ensure that failure will take place in flexure, not in shear?

Experiments have indicated that a lower limit for the ultimate nominal shearing stress, v, computed from Eq. 26 is about $0.05\ f_c'$. Accordingly a value $v \leqq 0.04\ f_c'$ may probably be used safely in ultimate load design as a limit for members without any web reinforcement. This is a radically low value, as the allowable *working* stress given by ACI 318-51 is $0.03\ f_c'$. Tests reported by Talbot, Clark and recent tests by the writer indicate that some beams without web reinforcement may fail in shear at nominal shearing stresses between 5 and 6 percent of f_c'.

Experiments have also indicated that the ultimate strength of beams with web reinforcement is at least that resulting from neglecting all contribution of the concrete.

$$v = \frac{V}{bjd} = \frac{A_v f_{yp}}{b\ s} (\sin \alpha + \cos \alpha) = Kr f_{yp} \quad \ldots\ldots (29)$$

The use of Eq. 29 as an ultimate load design equation is believed to result in members which will have some reserve strength in shear when failure in flexure takes place.

In European working load design, web reinforcement, wherever necessary, has, for several decades, been designed for the total shear. In some countries building codes even stipulate a minimum amount of stirrup reinforcement to be used in all beams however small the nominal shearing stress may be. Furthermore, the design philosophy that particularly undesirable modes of failure should be avoided entirely is common in European thought. For these reasons European supporters of ultimate load design seem to be well satisfied with Eq. 29.

Bond and anchorage

In a similar manner as for the case of shear and diagonal tension, few new basic findings pertaining to bond and anchorage have been presented in later years that were not known to the pioneers in the field. In 1913, D. A. Abrams discussed the presence of high local bond stresses at flexural cracks in the concrete. He outlined the desirable characteristics of a deformed reinforcing bar, and used the following equation for the average nominal bond stress.

$$u = \frac{V}{jd\ \Sigma o} \quad \ldots\ldots (30)$$

As deformed bars meeting ASTM designation A 305-50T are commonly used in this country and the highest yield point specified, that for hard grade and rail steel, is *only* 50,000 psi, bond stresses will hardly be a serious problem in ultimate load design. The use of Eq. 30 as an ultimate load equation with $u = 0.20\ f_c'$ for bottom bars and $u = 0.14\ f_c'$ for top bars is believed to result in members which will have a reserve strength in bond when failure in flexure takes place. Even the supporters of *truly balanced design* have never, to the writers knowledge, advocated the inclusion of bond failures in the group of equally probable modes of failure.

The bond between concrete and reinforcing steel serves three major purposes: to ensure anchorage of the reinforcement, to prevent a general slip between concrete and reinforcement even though local slip must be present at flexural cracks, and to limit the width of flexural cracks at service loads through holding down the spacing of such cracks. Limiting crack width may conceivably become the most serious bond problem in the future.

An introduction of ultimate load design with its inherent recognition of the strength contributed by high strength reinforcement may lead to future American manufacture of reinforcing bars with yield strengths approaching 100,000 psi. It may in this connection be noted that present Austrian specifications allow working stresses of 50,000 psi for deformed reinforcement with a minimum yield strength of 86,000 psi. With such working stresses even better bond characteristics than ASTM A 305-50T may be necessary to hold the crack width under about 0.01 in. which is believed by several European authors to be a safe upper limit for structures exposed to normal weathering.

CONCLUSION

Ultimate load design of reinforced concrete members is not a recent development. The work of pioneers in concrete design was directed primarily toward predicting the strength of members, not the stresses at service loads. A design code based on ultimate loads was suggested by a committee of the NACU as early as 1908, and the present ACI Building Code 318-51 is based directly on ultimate load equations as far as columns without or with small eccentricities are concerned.

Ultimate load design of reinforced concrete members must necessarily be based on facts established through experimental investigation. Expressions for the ultimate strength of some types of members, such as beams failing in flexure and concentrically loaded columns, have been very well established through a large number of tests. Though a smaller number of test results are available ultimate load equations for members subject to combined flexure and axial load have also been well established. Limited knowledge is available regarding more complex cases particularly those involving shear failures. In such cases where no reliable ultimate load equations are yet known, the straight-line theory will permit an analysis, but we do not know how to interpret such an elastic analysis intelligently in terms of safety against failure.

The procedures of ultimate load design must necessarily be based on experimental facts, but may also be considerably influenced by design philosophy. Two major groups of such judgment and opinion appear in literature concerning ultimate load design. One group of authors have asserted that the ideal reinforced concrete structure will result from a *truly balanced design*, characterized by several modes of failure being equally probable if the structure is accidentally overloaded. The writer cannot accept this basic philosophy, he must join the group supporting the view that particularly un-

desirable modes of failure should be made very improbable by purposely discounting the corresponding ultimate load equations. If a structure designed in this manner is accidentally overloaded, extensive cracking will give warning of distress, and failure will take place in a ductile manner, the energy absorbing capacity being relatively large. True, the structure may fail, cracks may open, concrete and plaster may spall, beams and columns may deflect badly; but the structure is not likely to collapse, and repairs may be possible.

Design philosophy has been emphasized throughout the present paper in the hope of stimulating the discussions necessary to clarify the basic thinking involved in design. The behavior and strength of reinforced concrete members may be studied by relatively few research workers, and the ultimate load design equations may be established on the basis of experiments with the aid of statistical methods. However, design philosophy can only be wisely formulated on the basis of the experience and judgment of concrete engineers as a professional group. A future discussion of basic thinking is therefore believed to be of great importance in establishing sound design methods which will result in structures performing the primary function intended in design, to carry loads as well as possible.

Discussion

By VERNON P. JENSEN*

INTRODUCTION

Professor Hognestad ably states the fundamental concepts of ultimate load design. He carefully separates factual data from matters of opinion and from design philosophy. He brings out some of the differences of opinion expressed by earlier writers, and takes a stand in favor of a particular philosophy of design. All this is good. The factual data are true and verifiable by the knowledge we have today. This factual information reduces to ultimate load formulas plus a knowledge of the behavior of reinforced concrete members all the way to failure. The formulas constitute the framework on which we may build a method of ultimate load design. The knowledge of concrete behavior constitutes a background of information absolutely essential to the proper setting of load factors and other code requirements.

Test data on ultimate loads have always constituted a sort of framework for design, but the framework has seldom been openly and clearly expressed in relation to design. For the most part the relationships between design and ultimate loads have been known to the laboratory researchers, to the code makers, and to a relatively few specialists in reinforced concrete.

*Member American Concrete Institute, Staff Consultant, C. F. Braun and Co., Alhambra, Calif.

Today ultimate load theories are advanced sufficiently far to yield reliable predictions of manner of failure and ultimate strength of members. Formulas for ultimate strength are now available in practical form for use in design. At the risk of repeating some of the things Professor Hognestad has already said, the writer would like to consider a few of the questions that come up frequently with respect to theories and formulas for ultimate strength, and with respect to their use in design.

IMPORTANCE OF STRESS BLOCK

First, are ultimate strength theories based primarily on the behavior of concrete as observed in cylinder tests? No, they are based primarily on tests of beams and columns under varied conditions of loading. Observations on cylinders serve only as a guide to help set the form or the properties of the stress block of concrete in flexure.

A great deal of attention has been given to the form of the stress block in flexure. Of what importance is this form? Almost none, so far as ultimate loads in tension are concerned, a great deal, in predicting the manner of failure and a great deal in predicting ultimate loads when the initial failure is by compression.

Are we certain of the form of the stress block in flexure? By no means. However, this is not a serious limitation to ultimate load theories and formulas. The important fact is that empirical constants, such as those now appearing in ultimate formulas, are capable of producing reasonable agreement with all known flexural tests and with all known tests of eccentrically loaded columns.

The emphasis given to the stress-strain relation of concrete beyond the maximum stress brings up another question. Does this emphasis indicate an intention to use such highly strained concrete as an engineering material? It does not. But it does indicate the need for using this portion of the stress-strain diagram to explain observed ultimate strengths.

SELECTION OF SERVICE LOADS

Would the adoption of ultimate load design alter the designer's responsibility for selecting service loads? No, service loads are synonymous with design loads as now used. Load factors should be specified so that we can apply them to the design loads we are now using.

Would ultimate load design produce structures having a greater tendency to crack and deflect under service loads? Not if load factors and other code requirements are properly set. If the load factors are made too low, the resulting structures will deflect excessively and they could be dangerous. If the load factors are made too high, the resulting structures will be uneconomical and excessively strong. Between these extremes lies a selection of load factors that would result in satisfactory and economical design.

Suppose the method of ultimate load design were adopted. Would the designer then have any latitude in choosing formulas for ultimate strength?

Probably not. Formulas for ultimate strength would have to be specified in codes and building regulations. It is doubtful that alternative formulas would or should be included in these regulations.

PERCENTAGE OF TENSION REINFORCEMENT

In the use of ultimate load design, would it be desirable to have a code restriction on the maximum percentage of tension reinforcement to be permitted without compression reinforcement? In the writer's opinion, yes. Limiting the percentage of reinforcement is one way to avoid designs that could fail initially in compression from overload. Excessive deflections would be avoided as well. It is true that the percentage of reinforcement would generally be limited anyway by economics and by physical limitations on the crowding of tension steel. It would be prudent, at least in starting out with ultimate load design, to put code restrictions on percentage of reinforcement and on depth of slabs.

A simple expression for maximum permissible tension reinforcement, without compression steel, is of the form

$$p_{max} = \tfrac{1}{8}\frac{f_c'}{f_{yp}} \qquad (31)$$

The constant in this equation is approximately that applying to Professor Hognestad's Fig. 4 at the intersection of the lower curves. In advocating a limiting percentage of tension steel, the writer agrees with Professor Hognestad's philosophy of design. If there are any differences, they are simply in details of the method of putting this philosophy into practice.

Title No. 48-54

Review of Research on Ultimate Strength of Reinforced Concrete Members*

By C. P. SIESS†

SYNOPSIS

Extensive and sometimes comprehensive research on the ultimate strength of reinforced concrete members has been carried on almost since the first use of this structural material. This paper reviews briefly some of the more significant work in this field, both experimental and analytical. The scope is limited to statically determinate isolated reinforced concrete structural elements, divided for the purposes of discussion into five categories depending on the predominant type of stress to which they are subjected: (1) pure flexure, (2) axial compression, (3) combined axial compression and flexure, (4) combined flexure and shear, and (5) combined flexure, shear and axial compression. The paper is not intended to be comprehensive; only those tests and theories are discussed that serve best to give an over-all picture of the development and scope of research on ultimate strength of reinforced concrete members.

INTRODUCTION

Research on ultimate strength is by no means a recent development. From the very first, tests made on reinforced concrete members involved studies of ultimate strengths and modes of failure, and attempts to develop theories capable of predicting these phenomena followed quite naturally in the early stages of research. By 1900, or shortly thereafter, the phenomena of failure and ultimate strengths of beams in flexure and columns in axial compression were fairly well understood, and theories for ultimate strength had been developed. Tests and theories for eccentrically loaded columns were to come somewhat later, while the ultimate strength of beams failing in shear even now cannot be predicted with assurance.

All present knowledge regarding the ultimate strength of reinforced concrete members is based on the results of tests, and any theory of ultimate strength is limited in its applicability by the scope of those tests. The object of this paper is to review briefly the tests of all kinds that have been made on the various types of reinforced concrete members, to consider their number and scope, and to show the part they have played in the development of ultimate theories.

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†Member American Concrete Institute, Research Associate Professor of Civil Engineering, University of Illinois, Urbana.

SCOPE

This paper is concerned only with the ultimate strength of isolated members, not with the strength of structures composed of such members. For this reason, tests of frames, arches, or other indeterminate structures have not been included, since the forces acting on the elements of such structures cannot be determined directly from the applied loads, especially at or near failure. Similarly, tests of isolated slabs, which are internally indeterminate, have been omitted.

As an aid to discussion, it is convenient to classify the numerous tests of reinforced concrete members in some orderly fashion. For this purpose two characteristics of the tests can be recognized: (1) the predominant type of stress resisted by the member, and (2) the time characteristics or frequency of application of the loading.

For the stress classification, five types of stress or combinations of stress can be considered: (a) flexure, (b) axial compression, (c) combined axial compression and flexure, (d) combined flexure and shear (due to bending and/or torsion), and (e) combined flexure, shear, and axial compression.

The most common type of loading in tests, although not necessarily always the most important from the standpoint of service conditions, is the common short-time "static" test in which the load is applied progressively until failure occurs, the test usually being completed in a length of time ranging from a few minutes to two or three hours. Probably nine-tenths of the tests made on reinforced concrete members have been of this type. However, at least four other types of loading have been used or should be considered as representative of conditions in service. They are:

(a) Sustained working loads (creep test)—Of interest to the extent that deformations occurring in such a test affect the ultimate strength.

(b) Sustained high loads—Tests to determine the maximum load that can be sustained indefinitely by a member.

(c) Dynamic loading—Occurring in service usually as the result of rapidly moving loads, earth movements, or air blast. In the latter two cases the energy-absorbing capacity of the member may be of as much or greater interest than its ultimate strength.

(d) Fatigue or repeated loading.

MEMBERS SUBJECTED TO PURE FLEXURE

Development of ultimate theories

Although reinforced concrete members are seldom, if ever, subjected to a state of pure flexure in actual service, the ultimate strength for this type of loading has been the object of continued research since the development of reinforced concrete as a structural material. The case of pure flexure is of fundamental interest. Although in practice flexure is usually accompanied by shear or axial compression, or both, a knowledge of the behavior and strength of a member in the pure state of stress is essential to an understanding of its behavior when subjected to more complex stress conditions.

The first ultimate theory for beams in flexure reported in the literature was probably that advanced by Koenen in 1886[1] and based on tests made by

Wayss and Koenen reported in the following year.[2] This theory and others like it were undoubtedly used as the basis for design of beams prior to the introduction of the "straight-line" theory about 1900. Refinements were made in the theories as the results of further tests, and the inelastic action of concrete at the ultimate was recognized in von Thullie's theory of 1897[3] and Ritter's of 1899.[4] The first theory of this type in America was Talbot's familiar parabolic theory, presented in 1904[5] which was essentially the same as Ritter's.

Although new theories were presented from time to time, no real advance was made until the early 1930's when research in this field was given new impetus by a paper by Emperger in 1931.[6] Shortly after this, in 1932, Stüssi in Switzerland presented an ultimate theory having several unique features.[7] Those properties of the stress block affecting the ultimate strength, rather than the shape of the stress block itself, were considered, leading to a theory ideally suited to the empirical approach characteristic of research in reinforced concrete. Stüssi also included the ultimate strain in the concrete as a factor affecting the strength of a beam failing in compression. Although Stüssi based his ultimate strains on those obtained in tests of axially loaded cylinders, Baumann in 1934[8] and Brandtzaeg in 1935[9] found that substantially higher maximum fiber strains could be developed in flexural compression than in cylinder tests. This had been noted by Talbot as early as 1904,[10] but was not introduced into his theory. The introduction of concrete strain as a major variable emphasized the need for considering the characteristics of the concrete stress block as determined from tests of beams rather than from tests of cylinders. The theories developed by Brandtzaeg in 1935,[9] Jensen in 1943,[11] and Chambaud in 1949[12] all followed this line of attack, and all may be considered modern theories. Even yet, however, there is by no means complete knowledge regarding the various properties of the stress block in a beam.

In America, interest in ultimate theories was revived in 1937 by Whitney's paper presenting an ultimate theory based on a rectangular stress block and factors determined empirically from tests.[13] This relatively simple theory was proposed for use in design, and was extended to include also the case of combined flexure and axial compression. In 1941, Cox[14] presented a similar theory, and in 1943 Jensen[11] reported the development of a theory, similar to that of Brandtzaeg, in which the character of the stress block was related to the strength of the concrete.

Tests

Except for those few intended to provide simplified design procedures, all of the ultimate theories were developed to predict the behavior and ultimate strength of a beam in flexure. Specifically, an ultimate theory should be able to predict whether a given member will fail in tension or compression, and the ultimate strength in tension or compression, as the case may be. Since all of the theories now available were derived from the results of tests, we can reasonably expect them to be capable of predicting ultimate strengths,

at least for the range of conditions and variables present in the tests on which they were based. Whether they are generally applicable to all types of loadings and over a sufficiently large range of variables will depend on the breadth of their experimental foundation. It is appropriate, therefore, to look at the tests.

Most, if not all, of the ultimate strength theories for members subjected to pure flexure have been based on static tests of simply-supported beams loaded at two points located symmetrically about midspan. For this condition of loading, the portion of the beam between the loads is subjected only to flexure. Only those beams failing in this region will be considered; failures at other locations are due to a combination of shear and flexure.

The first really engineering tests on reinforced concrete members were probably the tests on beams reported by Wayss and Koenen in 1887.[2] The pioneering work in this country included the tests made by Talbot[5,10,15,16,17] and Withey[18,19] in the period from 1904 to 1908; and the extensive series of tests reported by Humphrey and Losse in 1912.[20] These tests totalled about 500 and constitute probably half of all the tests on beams made in the United States.

After about 1912 there seemed to be a lull, which lasted until 1930 when the testing of beams was again resumed in full force. During the 1930's and early 1940's tests of perhaps 300 beams were reported.[14,21-27]

Although a complete census has not been made, it appears that well over 1000 reinforced concrete beams have been tested to failure under the conditions mentioned in the preceding paragraph. Of these, probably three-quarters or more have failed in tension, the others in compression. The predominance of tension failures reflects the greater attention given by investigators to the behavior of beams designed by conventional procedures.

For failures in tension, the results of the tests are usually in good agreement with the predictions of most theories. This is to be expected, since the various assumptions that have been made in developing the theories have only a relatively small effect on the predicted ultimate strength for tension failures; even the ordinary "straight-line" theory is reasonably accurate in this respect.

Since beams designed by conventional procedures usually fail in tension, the strength in compression has been of interest primarily in connection with the development of ultimate theories. In the United States, only about 200 compression failures have been reported, and nearly all of these have been utilized in deriving or checking the ultimate theories of Whitney, Cox, and Jensen.[14,20-26]

Test results suitable for determining the boundary between tension and compression failures are available mainly from three series of tests made in this country, involving a total of 197 beams divided roughly equally between compression and tension failures.[14,26,28]

The major variables affecting the flexural strength of beams in static tests are the shape and dimensions of the beam, the ultimate strength of the concrete and phenomena associated with it, the yield strength and stress-strain

characteristics of the reinforcement, and the amount and location of the reinforcement. The known tests have included all of these variables, for the most part varying over a wide range. Nevertheless, the number of tests of beams with compression as well as tension reinforcement failing in compression is rather small, and may be considered not in keeping with the great frequency with which this type of beam is used. This deficiency is partly removed, however, by the information obtained in tests of members with combined flexure and axial compression.

Loadings other than short-time

So far as the writer knows, all of the ultimate theories for beams in flexure have been developed on the basis of data obtained in ordinary short-time static tests to failure. Other types of loading, having different time characteristics, which may be of interest in design include sustained high loads, sustained working loads, fatigue or repeated loading, and dynamic loading.

Except for a few tests of relatively short duration made by Talbot in 1905,[5] the writer knows of no systematic or extensive investigation of the ability of beams to sustain loads near their short-time ultimate strength for an indefinite length of time. Research on this subject would seem to be of particular interest, especially in the case of compression failures, in view of the known time-deformation characteristics of concrete.

The significance of creep tests in relation to ultimate strength lies in the possible effects that the deformations occurring under sustained load may have on the strength of the member when tested to failure statically. No tests on simple beams made to determine these effects are known to the writer. However, a few tests of reinforced concrete frames, loaded first under a long-sustained load and then tested to failure statically, have suggested that the ultimate strength for failure in tension is not affected by the prior creep deformations.[29,30] No evidence seems to be available regarding the effect of creep deformations on the ultimate strength for compression failures or on the maximum deformation that the beam can undergo before failure.

Fatigue tests enter the picture in somewhat the same way as creep tests. If the repeated loads are large enough to cause failure after a finite number of repetitions, it is the ultimate strength in fatigue that governs design. However, if the magnitude of the repeated load is less than the endurance limit of the member, failure in fatigue will not occur under an indefinite number of load applications. In this case, we are concerned primarily with the possible effects on the ultimate strength as determined from subsequent short-time tests. About 250 fatigue tests of beams are known to the writer, but not all of these involved failures in flexure, and very few were conducted in such a manner as to determine the effect of many repetitions of a low load on the ultimate strength.[31] The data available regarding the strength in fatigue are very limited and the results show much scatter. However, the few data available indicate that the endurance limit for failure in pure flexure is in the neighborhood of 50 to 60 percent of the short-time ultimate. Data

on the number of cycles necessary to produce failure at loads above the endurance limit are too fragmentary to be useful except as a guide to further research.

Also of interest are tests in which a load relatively near the ultimate is applied slowly a sufficient number of times to determine whether the inelastic deformations occurring at high loads increase progressively or stabilize under repeated application of load. A few tests of this type were made in the early stages of research in reinforced concrete,[5,16] but this aspect of the ultimate strength problem is still relatively unexplored.

All dynamic load tests of reinforced concrete beams known to the writer were made with a single load applied at some point on the beam. For this condition, shear is present in all regions of the beam and no data on strength in pure flexure are available.

Summary

It should be evident from the foregoing discussions that the ultimate strength theories for beams failing in flexure are well established on a firm foundation of experimental evidence, covering practically all of the known variables except perhaps certain types of loading other than static. Moreover, even in these cases, our understanding of the behavior of members in flexure is sufficient to provide a sound and safe basis for design.

MEMBERS SUBJECTED TO AXIAL COMPRESSION

Historical review

Members subjected to axial compression have been represented in tests by axially-loaded columns. Although this type of load is probably never encountered in actual construction, its study is essential to an understanding of the behavior of columns, and it represents a limiting case for the column subjected to both axial load and bending.

The axially-loaded column is probably unique among reinforced concrete members in the number of tests that have been made. Well over 2000 such tests have been reported and, as a result, our knowledge of the behavior of axially-loaded columns undoubtedly has a firmer basis than that for any other type of structural members.

Reinforced concrete columns are of two basic types: (1) "Tied" columns, in which the longitudinal reinforcement is encircled at intervals by steel ties, and (2) "spiral" columns, in which the reinforcement is contained within a continuous helix of wire or small-diameter bar, the so-called "spiral" reinforcement.

The ultimate theory for tied columns is a simple addition law, stating that the ultimate strength is the sum of the strengths of the longitudinal reinforcement and the concrete in the column. This law was stated by Considère in his book, an English translation of which was published in 1903.[32] Research since that time has been concerned primarily with determining the relation between the strength of the concrete in a column and the ultimate compressive strength of standard test specimens. Much research has

also been done to determine the validity of the addition law for a wide range of steel and concrete strengths, percentages of reinforcement, and stress-strain properties of the steel, particularly for steel without a well-defined yield point.

The action of the spiral reinforcement was also studied by Considère around 1900, and the effects of spiral reinforcement on the ultimate strength and behavior of a column were clearly stated by him.[32] However, the fundamental studies by Richart, Brandtzaeg, and Brown in 1928–29 provided a more complete and accurate explanation of the mechanism of spiral action, and provided extensive quantitative data from which the contribution of the spiral to the ultimate strength could be predicted more accurately.[33,34]

Undoubtedly, the most extensive investigation of axially-loaded columns, and the one having the greatest effect on design practices, was the ACI Column Investigation, carried out simultaneously at the University of Illinois and Lehigh University in the 1930's.[35–45] At the time this investigation was begun, a total of about 1000 tests of axially-loaded columns had been reported, divided about equally between the United States and Europe, and nearly all made prior to 1916.[35] The ACI Column Investigation included tests of 582 columns, more than had been tested in the United States up to that time. Since the ACI Investigation, tests of an additional 400 or more columns have been reported,[45–49,51] bringing the total number to over 2000—truly an impressive figure.

Variables

The principal variables affecting the ultimate strength of an axially-loaded column have been investigated over a wide range. In the ACI Column Investigation alone, the concrete strength was varied from 2000 to 8000 psi, and both moist and dry storage conditions were included; the percentage of longitudinal reinforcement was varied from 0 to 6 percent, and steels of structural, intermediate, and rail grades were used; the percentage of spiral reinforcement was varied from 0 to 2 percent, and spirals consisting of both mild steel and hard-drawn wire were tested. Other investigations have included even greater percentages of longitudinal reinforcement than the 6 percent mentioned above,[50] and tests have been made with steels having very high strengths[45] and having stress-strain characteristics different from those for mild steel.[51]

Although most of the early spiral columns were tested without shells, the tests reported by Richart in 1947–1948 included 168 columns having a large range of shell thicknesses and ratios of shell area to the area of the core.[48,52]

The great majority of the columns tested were rather short, having a ratio of length to width between 5 and 10. However, tests of columns having much greater slenderness ratios have been reported by Thomas,[47] Baumann,[8] Coyne[46] and Rambøll[49] in Europe, and by Richart[48] in this country. Slender spiral columns have usually failed by buckling in the spiral range because of the low effective modulus of elasticity resulting from the large deformations required to develop the strength of the spiral. The failure of such columns

is thus due to a combination of axial load and bending, and ultimate strengths can be predicted by means of a "reduced-modulus" theory. Long tied columns, on the other hand, have usually failed by crushing for values of l/d as high as 20, and the reduction in ultimate strength below that given by the addition law for axially-loaded columns is usually small.

Load characteristics

By far the greatest amount of information regarding the ultimate strength of axially-loaded columns has been obtained from short-time tests; probably 90 percent of all the tests made have been of this type.

Series 3 of the ACI Column Investigation involved tests of 128 columns, both tied and spiral, under sustained working loads, and stored either in a moist room or in the air of the laboratory.[38,39,42,43,45] These tests furnished much valuable data on the redistribution of stress between the concrete and the reinforcement that takes place as a result of creep and shrinkage of the concrete. However, of more importance in connection with ultimate strength are the results of short-time tests to failure of some of these columns that had been held under a sustained load for one year. Although all of the columns tested in this manner had undergone considerable redistribution of stress, their ultimate strengths after a year of sustained loading were equal to those of companion columns which had not previously been loaded. This result is consistent with the known ability of the reinforcement to deform plastically after it reaches its yield point and thus permitting the concrete to develop its ultimate capacity, no matter how much the steel was "prestressed" under the sustained loading.

Relatively little is known regarding the ability of axially-loaded columns to sustain high loads near their short-time ultimate for an extended period, and this aspect of column behavior has played no part in the development of ultimate strength theories. Series 2 of the ACI Column Investigation included a comparison of ultimate strengths obtained in ordinary short-time tests and in "slow" tests.[36,37,45] The "slow" test consisted of loading the column to three-fourths of its ultimate capacity at the usual rate and then applying the remaining load in about 10 increments at intervals of 4 hours. A total of 159 plain, tied, and spiral columns were tested at the two rates of load application, and no significant difference between the ultimate strengths in "slow" and "fast" tests was noted. However, these tests cannot be taken as conclusive proof that the short-time strength can be resisted indefinitely since the duration of the "slow" tests was only about two days.

Series 4 of the ACI Column Investigation, carried out at Lehigh University, included tests of 16 columns under high loads sustained until failure occurred.[44] The test loads ranged from 70 to 95 percent of the average ultimate strength of three companion columns loaded to failure in short-time tests. For load percentages above 85 percent, failure usually took place in a few days or even in a few minutes. However, below 85 percent of the short-time ultimate load, no failures had occurred after as long as two years in some cases. Four of the tests were made on tied columns, while the re-

mainder were on spiral columns for which the sustained loads were well into the spiral range. All of the spiral columns failed by lateral buckling, although some of them were still holding load after one year even though they had buckled and spalling of the concrete was in evidence. Additional information is still needed regarding the sustained-load strength of tied columns and of spiral columns loaded not in excess of their yield point.

No fatigue tests of axially-loaded reinforced concrete columns are known to the writer, although many tests have been made to determine the fatigue strength in axial compression of plain concrete cylinders or prisms. However, in the early days of research in this country, it was fairly common practice to make tests in which the load—frequently a fairly high load—was repeated a few tens of times at the rate usual in short-time tests, and a few tests of this type are available from the early work of Withey[53] and Talbot.[54]

No tests of axially-loaded columns under impact or other dynamic loading are known except the tests on precast reinforced concrete piles made by Glanville and others in England.[55] These tests consisted, in effect, of repeated impact tests—a type of interest in certain applications, but one very difficult to interpret in view of our present limited knowledge of behavior in both impact and fatigue.

MEMBERS SUBJECTED TO COMBINED AXIAL COMPRESSION AND FLEXURE

The combination of axial compression and flexure is the first stress condition to be considered in this paper that is typical of conditions occurring in actual members. Eccentrically-loaded columns, members in framed structures, and arches are all typical members which in service must resist stresses due to both axial compression and flexure.

All of the tests made to determine the ultimate strength of members subjected to pure flexure or to axial compression, and discussed in the preceding two sections, have application to this problem. These tests represent the limiting conditions for members subjected to combinations of axial compression and flexure, and provide boundary conditions which must be satisfied by any ultimate strength theory for such members. Although the existence of ultimate theories for the limiting cases is of great help to the investigator, the problem which remains is to determine the interaction of the two types of stress over the entire range of possible combinations, and to determine its effect on the ultimate strength and mode of failure.

Tests

Experimental studies of combined axial compression and flexure have nearly all involved tests on eccentrically-loaded columns, for which load was applied along a line parallel to the axis of the column.

In the United States, relatively few tests on eccentrically-loaded columns were made prior to 1936. These included tests reported by Talbot in 1907[54] and Withey in 1910-1911.[53,56] Other tests made by Talbot and Richart in 1916 and 1925 were not reported until 1947.[48] This group of tests totalled

about 55, all made prior to 1925. However, since that time, tests of 432 eccentrically-loaded columns have been reported in this country. These include: Richart and Olson, 1938,[48,57] 82 columns; Andersen, 1941[58] and 1951,[59] 230 columns; and Hognestad, 1951,[60] 120 columns.

Many additional tests have been reported by European investigators. They include: von Thullie, 1909,[61] 480 columns; Bach and Graf, 1912-13,[62] 51 columns; Brandtzaeg, 1936,[9] 13 columns; and Thomas, 1938,[63] 55 columns.

The tests mentioned in the preceding paragraphs total 1085. Practically all major variables were included: both tied and spiral columns; spiral columns with and without shells; round, square, and L-shaped cross sections; various concrete strengths; and various amounts and arrangements of reinforcement. The magnitude of the eccentricity of the load was a major variable in all of the tests, and failures by yielding of the reinforcement as well as by compression of the concrete were obtained. With only one exception, all of the tests were made with the bending applied in the plane of an axis of symmetry of the cross section. However, in the tests reported by Andersen in 1941, some 70 of the tests were made with the load applied on a line making an angle of 22.5 degrees with the diagonal of a square cross section. In the other tests of this series the load was applied either on the diagonal or on a line equidistant from two faces of the column. This last-mentioned condition was used also in most of the other tests on square or rectangular columns.

Ultimate theories

Prior to the middle 1930's the presence of flexural stress was usually neglected in the design of columns in framed structures. Where it was necessary to consider the effects of flexure, stresses for working loads were computed on the basis of the straight-line theory, using the transformed section, and neglecting tension in the concrete. This procedure, of course, did not involve the ultimate strength of the member. However, in 1938, an expression was presented by Richart and Olson[57] which related the ultimate load on an eccentrically-loaded column to that for an axially-loaded column thus:

$$\frac{P_{ecc.}}{P_{axial}} = \frac{1}{1 + C\,\frac{ec}{k^2}}$$

Where e is the eccentricity of the load with respect to the centroid of the column cross-section, c is the distance from the centroid to the extreme fiber in compression, k is the radius of gyration of the column section, and C is an empirical constant less than unity. This was an empirically derived ultimate strength formula based on the results of tests by Richart and Olson as well as those by Thomas, Bach and Graf, and von Thullie. The expression given can be interpreted as stating that the ultimate strength depends on the maximum fiber stresses in compression, resulting from both the axial load and bending, and assuming a linear distribution of stress across the section. However, since it was known from tests of beams in pure flexure that flexural

compressive stresses computed on a straight-line basis could attain much higher values than the 0.85 f_c' commonly used as the ultimate strength in axially-loaded columns, the equation given involved the constant C which had a value less than unity, and thus discounted the contribution of the flexural stress.

This equation, being based on extensive tests, was empirically sound and was capable of predicting with considerable reliability the ultimate strengths for failures in compression. However, by its nature and derivation, it was not capable of predicting the ultimate strengths for failures by yielding of the reinforcement or of predicting whether a member would fail in this manner or by crushing of the concrete. For this purpose, a theory more accurately reflecting the real behavior of the member was required, and such a theory, based on nonlinear stress-strain relation for concrete was presented by Brandtzaeg in 1935.[9] Since this theory took into account the inelastic behavior of the column, it was capable of predicting the behavior over the full range of action from axial compression to pure flexure, although different expressions for ultimate loads were necessary for failures in tension and in compression.

Whitney in 1937 and later presented ultimate theories for compression and tension failures of members subjected to axial compression and flexure.[13,64-66] For compression failures his expression reduced to that for axially-loaded columns under certain conditions, while, for tension failures, the expression reduced to that previously presented by him for beams subjected to flexure alone. Data from the tests by Bach and Graf, Richart and Olson, and Thomas were compared with the predictions of the theory, with fair agreement.

In this country two attempts have been made to develop ultimate strength theories for eccentrically-loaded columns based on the inelastic properties of the stress-strain curve for concrete. One of these, utilizing the stress-strain relations assumed by Jensen, was developed by Boase and others in 1944.[67] The other was developed by Hognestad in 1951[60] as the result of an extensive series of tests, and involved a rather complex, but realistic, stress-strain relationship for the concrete.

The several theories mentioned differ but little in their ability to predict the ultimate strength for tension failures since the assumed compressive stress distribution is of little importance for this case. However, some are much better than others in their ability to predict strengths for compression failures or to predict whether a column will fail by tension or compression. In general, the more realistic, and thus more complex, assumptions regarding the stress-strain relation for concrete lead to the more widely applicable theories. For example, the theory presented by Hognestad, based on a stress-strain curve derived empirically from his tests, can predict not only the manner of failure and ultimate strength but also the behavior of the column throughout the entire range of loading.

All of the inelastic theories have two points in common; they involve different expressions for compression and tension failures, and these expres-

sions are much more complicated than those previously found for axial compression or pure flexure.

It is of interest, particularly in connection with the applications of ultimate theories to design, to note the use that has been made of interaction diagrams as a means of representing the ultimate theories for combined axial compression and flexure, and as a means of comparing those theories with the results of tests. The first use of an interaction diagram in connection with reinforced concrete columns was made by Thomas in 1938,[63] while the first use in this country was by Whitney.[65] More recently, in 1951 Hognestad[60] introduced the dimensionless interaction diagram similar to that used by Shanley for metal columns.[68] In this diagram, the quantities are M/M_o and P/P_o, where M and P are the applied moment and axial load, M_o is the ultimate moment capacity for compression failure in pure flexure, and P_o is the ultimate capacity of the column under axial load. With these parameters, all compression failures in the tests by Hognestad, involving numerous variables, could be represented by a single line on the interaction diagram. Tension failures, however, required separate lines for each group of columns having the same ultimate strengths for failure by tension in flexure.

Since reliable ultimate strength theories are available for the limiting case of axial compression or pure flexure, and for both tension and compression failures in the latter case, the remaining problem is that of determining the manner in which the strength varies between these two limits. For this purpose, the use of dimensionless interaction diagrams would seem to be a powerful tool and one which can provide ultimate theories which are both simple and accurate, especially for the case of columns failing in compression, which includes a large proportion of the columns most commonly found in framed structures.

Slender columns

Some mention should be made of the slender eccentrically-loaded column, although relatively little work has been done in this field, and none in this country. Both axial compression and flexure are present as in the case of the short columns, but, in this case, the flexure is due in a large measure to increases in eccentricity of load due to lateral deflection or buckling of the column. This problem requires extensive and rather complex analytical studies as a basis for interpreting the results of tests. Work in this field has been done by Baumann in Switzerland in 1934,[8] and by Hanson and Rosenström in Sweden,[69] and tests on about 53 columns have been reported in these investigations. Some work has also been done by Thomas in England (1939) in connection with tests of long axially-loaded columns.[47] In his studies of the test results, Thomas considered the eccentricities due to initial crookedness of the column. However, much more work remains to be done before the behavior of long eccentrically-loaded columns is clearly understood.

Loadings other than short-time

So far as the writer knows, no tests of eccentrically-loaded columns, or

other types of isolated members subjected to both axial compression and flexure, have been made except under the usual short-time "static" loading.

MEMBERS SUBJECTED TO COMBINED FLEXURE AND SHEAR

Practically all reinforced concrete beams are subjected to shear as well as flexure. The experimental work on this stress combination has usually involved tests of beams with one- or two-point loading. In the latter case, only the region outside the load points is subject to shear.

Failures of beams subjected to both flexure and shear may occur in flexure, in shear, or in bond. These three conditions are discussed separately in the following paragraphs.

Ultimate strength in flexure

The ultimate strength of beams failing in regions of pure flexure has been discussed previously. However, since shear is nearly always present in actual beams, it is necessary to ask how the ultimate strength in flexure is affected by the presence of shear. Although this question has received little consideration in the United States, a portion of the several hundred tests made in Europe have been directly concerned with this phase of the problem.

The European research on members subjected to shear as well as flexure has been aimed primarily at answering the question: How can beams be designed so that final failure will be by flexure rather than by shear and so that the ultimate strength and deformation in flexure will be comparable to that obtained in beams where flexure alone is present?

As a result of extensive tests by Mörsch,[70] Bach and Graf,[71-74] Saliger,[75] Emperger,[76] and others,* beginning in about 1903, European engineers have reached the conclusion that, in beams requiring web reinforcement, only by proportioning the web reinforcement to resist all of the stresses due to shear can the full flexural strength and deformation be developed and the absence of shear failure be assured. This finding, based on tests of typical beams, provides the best known basis for designing flexural members with assurance that the ductile type of flexural failure will be the only probable one. However, since European practice leans rather heavily on the use of bent-up bars as web reinforcement, while stirrups appear to be more popular in this country, it is not certain whether the conclusions reached by the European investigators can be applied unqualifiedly to conditions common in the United States.

Research in the United States

Research in this country has had as its main object the determination of the ultimate strength of beams actually failing in shear. To this end, most of the tests have been made on beams in which failure by flexure has been prevented by the use of large percentages of longitudinal reinforcement, thin webs, small span-depth ratios, or combinations of these factors. As a result, much of the available data on ultimate shear strength is for beams which are not representative of those commonly used in construction.

*See for example the references in "What Do We Know about Diagonal Tension and Web Reinforcement in Concrete? A Historical Study," by Eivind Hognestad. University of Illinois Engineering Experiment Station, Circular Series No. 64, 1952.

Tests for strength of beams in shear fall into two major categories: beams without and with web reinforcement. The latter group can be further subdivided on the basis of the type of web reinforcement used: bent-up bars, inclined stirrups, vertical stirrups, or combinations of these types. Because the case of beams with web reinforcement involves the greatest number of variables, many more tests have been made for such beams than for the case of beams without web reinforcement.

Another classification of tests in shear can be made: simple beams, and beams having loaded cantilever overhangs intended to produce a point of contraflexure in the region subjected to shear. The latter type of test has been made to simulate the conditions occurring in continuous or restrained beams. However, all such beams tested have actually been loaded in such a way as to be statically determinate.

Probably close to 1000 beams have been tested in this country in investigations intended to study the strength in shear. Not all of these beams have failed in shear, however; some have failed in bond, and some in flexure. Approximately one-third of the beams tested had no web reinforcement and the majority of these were included in tests made prior to 1922. The remainder were reinforced with either stirrups or bent-up bars or both, although very few tests with bent-up bars have been made since 1917.

The work on shear strength of beams in this country can be divided into three rather well-defined periods, each involving tests similar in object and approach.

Tests of 1903–1908

The first period covered the years 1903–1908, and included tests of about 122 beams by Talbot at the University of Illinois,[5,16,17,77] and tests of about 226 beams by Moritz and Withey at the University of Wisconsin.[18,19,78] These tests were the pioneering research on shear strength in the United States. From their results came a relatively clear picture of the modes of failure of beams both with and without web reinforcement, and an understanding of the variables affecting the ultimate strength in shear. Strengths were generally stated in terms of the maximum value of the nominal shear stress, $v = V/bjd$, sustained by the test beams, and evidence was presented to show that the ultimate strength in shear depended on: (1) the compressive strength of the concrete, (2) the percentage of longitudinal reinforcement, and (3) the ratio of length to depth of the beam for a given type of loading. The contribution of web reinforcement was recognized and much emphasis was placed on its effect on the character of the observed failures. It was repeatedly stated that beams without web reinforcement frequently failed in an undesirably sudden and violent manner, while beams with web reinforcement were much tougher and failed in a more ductile manner.

Tests of 1910–1922

The second period of major research on shear, from 1910 to 1922, included two major test programs. One was the extensive series of tests on both

simple and restrained beams, made at the University of Illinois under Talbot's direction during the period 1910–1922 and reported in 1927 and 1928 by Richart and Larson.[79,80] This group of tests involved 218 beams, of which only 65 failed in shear while the others failed in flexure or bond. The other major test program was carried out in 1918–1919 for the Emergency Fleet Corporation and was reported in 1926 by Slater, Lord, and Zipprodt.[81] Practically all of these tests were made on thin-web, I-shaped beams, usually with relatively large amounts of web reinforcement.

Both of these investigations involved extensive and elaborate studies of web reinforcement. The Illinois tests included many varieties of stirrups and bent-up bars, and combinations of the two, with many values of spacing, angle of inclination, radius of bend, and other variables studied. The Emergency Fleet Corporation tests included primarily stirrups as web reinforcement, but involved studies of vertical, horizontal, and inclined stirrups, as well as numerous combinations of these. In both of these investigations, the major emphasis was placed on stresses in the web reinforcement. The truss analogy or its equivalent, for computing the relation between shear at a section and stress in the web reinforcement, was available as an analytical tool, and the strain gage was available as a means of measuring strains in the web reinforcement. Consequently, much attention was paid to the relationship between nominal shear stress $v = V/bjd$ and stress, f_v, in the web reinforcement, both computed and measured.

Two findings regarding the v–f_v relationship were significant. One was that the relationship predicted by the truss analogy did not hold at low stresses because of the tensile stresses carried by the concrete which were neglected in the theory. The other was that the shear strength of a beam was greater than could be accounted for by considering only the contribution of the web reinforcement at the yield point. It is rather difficult to evaluate the contributions of these studies to our knowledge. The EFC tests were primarily concerned with the strength of thin webs in shear rather than with the strength of beams in shear. They served to show, however, that extremely high shearing stresses could be developed with sufficiently large amounts of web steel, and that other sources than the web reinforcement itself contributed to the strength in shear. This source was indicated as being the concrete in the web of the beam and was a function of the web thickness. However, the expression for ultimate strength finally recommended was of the form

$$v = 0.005 f_v + r f_v$$

in which $v = V/bjd$, r is the percentage of web reinforcement, and f_v is the stress in the web reinforcement, presumably to be taken as the yield point in determining the ultimate strength in shear. This equation was stated by the authors to assume "that the concrete participated with the web reinforcement in resisting the shearing stresses, that it was equally effective for all thicknesses of web, and that the stresses carried by the concrete increased proportionally with increase in the shearing strength."

No general conclusions regarding ultimate strength in shear were drawn from the Illinois tests, partly because only about one-third of the beams actually failed in shear, but chiefly because these investigations were made primarily to study the effects of several variables on the stresses in the web reinforcement. Richart stated, however, that the v–f_v relation was best expressed by an equation of the form

$$v = C + rf_v$$

with the constant C varying from 90 to 200 psi and probably depending "upon the percentage of web reinforcement used and also on the quality of the concrete." These tests provided much information of value regarding the relative effectiveness of various kinds of web reinforcement including stirrups, both vertical and inclined, and both loose and welded, as well as bent-up bars in various arrangements and in various combinations with stirrups, and on the effects of spacing of stirrups or bent-up bars.

Both of the investigations mentioned, through extensive strain measurements on the web reinforcement, added much to our knowledge of the action of web reinforcement in resisting shearing forces, both at working loads and near failure. However, their contribution to the development of a complete ultimate strength theory for beams failing in shear was of secondary importance.

Tests of 1945–1951

The current stage of research on shear strength is characterized by the attempt to evaluate in quantitative terms the contribution of the various elements of a beam to its strength in shear. Moretto's tests in 1945[82] were intended to determine the contribution of the concrete and the web reinforcement and involved the following variables: concrete strength, f_c'; percentage of web reinforcement, r; inclination of web reinforcement, α; and to a minor extent the percentage of longitudinal reinforcement, p. As a result of tests on 44 simple beams with third-point loading the following expressions were presented:

Shear at yielding of web reinforcement

$$v_y = Krfy + 0.04\, f_c' + 5000\, p$$

Shear at ultimate

$$v_u = Krfy + 0.10\, f_c' + 5000\, p$$

where $K = (\sin \alpha + \cos \alpha) \sin \alpha$, and f_y is the yield strength of the web reinforcement.

The series of tests reported by Clark in 1951[83] included 62 beams and involved the following variables: concrete strength; percentage of web reinforcement; percentage of longitudinal reinforcement; and ratio of depth of beam, d, to shear span, a. The following expression for ultimate strength in shear was given by Clark:

$$v = 2500 \sqrt{r} + 0.12\, f_c' \left(\frac{d}{a}\right) + 7000\, p$$

A comparison of the expressions given by Moretto and Clark illustrates very clearly the incomplete state of our knowledge regarding the strength in shear. In part, this is due to a lack of sufficient test data involving all of the known variables. For example, the term d/a appears in Clark's equation, but not in Moretto's since it was not a variable in the latter's tests. Similarly, since all beams tested by Clark had vertical stirrups having the same yield strength, the terms K and f_y do not appear in his equation. There is, of course, also a marked difference in the way in which the term r appears in the two equations as well as in the numerical values of the several coefficients. And finally, both series of tests were made on beams which were, in general, greatly over-reinforced against flexure.

Obviously much experimental work remains to be done before an expression for the ultimate strength of a beam failing in shear can be presented, and in particular, attention must be focused on beams having dimensions and properties more nearly representative of those found in structures.

Torsion

Thus far, consideration has been given only to shearing forces and stresses resulting from loads producing transverse bending in a beam. However, shearing stresses may also be present as a result of torsional moments applied to the beam, usually through other members framing into it. This condition occurs commonly in spandrel girders having a slab framing into them, in interior girders for unsymmetrical loading conditions, and in various types of skewed structures. Obviously, the strength of a member subjected to flexure and to shear stresses due both to transverse load and torsion is of practical importance in reinforced concrete design. However, no research on members stressed in this manner is known to the writer.

Ultimate strength in bond

The presence of bond, or rather the absence of slip, between the concrete and the reinforcement is usually assumed to exist in all theories of the behavior of flexural members, either at working loads or at ultimate. It is of interest to note, however, that the existence of this condition is not essential in order for a beam to carry load. A beam having unbonded reinforcement, anchored only at the ends, is entirely capable of resisting moment, although its ultimate strength in either flexure or shear will undoubtedly be different, and usually less, than that predicted by ultimate strength theories based on tests with bonded reinforcement. We can thus distinguish between two aspects of bond failure: loss of bond or excessive slip in the regions of a beam subjected to flexure alone or to flexure and shear, and loss of bond or excessive slip in a region providing anchorage. So long as anchorage still exists at the ends of the reinforcement, bond failures at other locations will not in themselves constitute failure, if by failure we mean total loss of load-carrying capacity. However, such local bond failures may lead to excessive widths of cracks or to large deformations, and may contribute to premature failure in either flexure or shear. On the other hand, anchorage failures, though

final in themselves, are not likely to occur except as secondary effects following initial failure or imminent failure in shear or flexure.

Research on bond has been of two types. One has been concerned primarily with the *average* bond stresses developed in either beams or pull-out type specimens. In beams the average bond stress is computed from the usual formula as $u = V/\Sigma ojd$, while in pull-out or push-out specimens the average bond stress is computed as $u = P/\Sigma ol$, where P is the load on the bar and l is the length of embedment. This type of investigation, however, has not always yielded data on the magnitude of the ultimate bond strength since many of the test specimens, especially those with deformed bars and some with plain bars having hooks or long embedded lengths, have not actually failed in bond in the sense that the bar was pulled out of the concrete. For this reason, it has been the practice, in both beam and pull-out tests, to compute average bond stresses at some arbitrary value of slip of the bar at its free end. More recently, since free-end slips are frequently quite small for bars having well-designed deformations, recourse has been made to the calculation of average bond stresses at given values of the slip at the loaded end of the bar. It is difficult to interpret the results of tests of this type in terms of ultimate strengths in bond, since the relation between slip of the bar and failure in bond has not yet been stated quantitatively. Consequently, this type of investigation, particularly as it relates to modern deformed bars, must frequently be limited to providing comparisons of the bonding characteristics or the bond-slip relation for various conditions. The effects of numerous variables have been studied in this manner, including type of bar and surface condition, strength of concrete, length of embedment, bar diameter, depth of concrete beneath the bar, position of casting, and stress conditions in the surrounding concrete. The last-mentioned variable has been studied by means of pull-out tests in which the concrete is in compression; pull-out tests in which the concrete is in tension; and beam-type pull-out tests, in which the state of stress in the concrete is the same as that in a beam. Many hundreds of tests of these types have been made by investigators since reinforcement was first used in concrete. The extensive tests, involving hundreds of specimens, reported by Abrams in 1913[84] was without question the first comprehensive study of bond; in fact, little has been done since that time that cannot be considered an extension of his thorough study. Another important contribution was the work reported by Gilkey, Chamberlin, and Beal in 1938, in which it was shown that the bond stresses developed were not proportional to concrete compressive strength above certain limiting values.[85] Menzel's studies in 1939 and 1941 of the effects of settlement of various depths of concrete beneath reinforcing bars required the introduction of a new factor into concepts of behavior in bond.[86,87] This, together with the development of new and improved types of deformed bars in the period 1941–1947, required a complete re-evaluation of the available quantitative data, and the work of Clark at the Bureau of Standards (1947–1951) has provided most of the data now available regarding the

bonding characteristics of current types of deformed bars.[88] These tests included both pull-out and beam specimens, and included as variables the type of deformation, bar diameter, length of embedment, depth of concrete beneath the bar, and strength of concrete.

The other type of research in bond, which is in some ways more pertinent to the development of ultimate strength theories, is that intended to provide a better understanding of the real phenomena of bond failure in a beam. This requires knowledge of the actual bond stresses and their distribution rather than the average bond stress. This problem was discussed by Abrams in 1913 in much detail and with insight gained from numerous tests. It was further discussed and elaborated by Mylrea in papers presented in 1926[89] and 1948,[90] and the most recent work along these lines is that reported by Mains[91] and by Bichara,[92] both in 1951. In the last two investigations mentioned, extensive use was made of short gage length electrical wire-resistance strain gages to determine the variations in stress in the bars and thus the variations in bond stress. These investigations have confirmed entirely the phenomena described qualitatively by both Abrams and Mylrea, and have contributed much to our knowledge of the phenomena of bond failures. However, sufficient information is not yet available to predict reliably the strength of a beam in bond under any of a wide range of possible conditions: that is, an ultimate strength theory for bond does not yet exist. On the other hand, the results of the investigations which have been made suggest that perhaps the relatively crude empirical data that we now have regarding bond stress is sufficient to permit the design of flexural members that will fail, at least initially, in flexure or shear rather than in bond. Nevertheless, since integrity of bond is probably essential to the development of full strength and deformation in flexure, more study of bond phenomena for conditions similar to those in a beam near failure is needed.

Loadings other than short-time

As might be expected, most of the tests of beams subjected to both flexure and shear have been the usual short-time static tests. However, an appreciable number of tests have been made with other types of loadings and a major portion of these have been concerned with strength in bond.

Tests of beams under both low and high sustained loads for the purpose of producing failures in bond were reported by Abrams in 1913,[84] by Brown and Clark in 1931,[93] and by Turley in 1915.[94] Pull-out or push-out tests under sustained loads, both low and high, were reported by Davis, Brown, and Kelley in 1930,[95] and by Shank in 1931 and 1939.[96,97] Only the tests by Turley and by Brown and Clark included deformed bars, and it is questionable whether any of these are applicable to bars of the types currently used. The same question may be raised regarding the data from tests with plain bars in view of recent findings suggesting changes in the surface conditions of such bars from those existing 20 years ago.[86]

Fatigue tests to study bond in beams were reported by Withey in 1909.[98] However, their application to present day materials is doubtful. Tests of

pull-out specimens in fatigue have been reported by a number of persons, the most recent being those by Muhlenbruch in 1945 and 1948.[99] So far, however, our knowledge of the fundamentals of behavior in bond have not permitted use to be made of data from fatigue tests.

Most impact tests on beams have been made with a single load applied at some point along the span, usually at midspan. Drop tests on beams were reported by Mylrea in 1940,[100] and tests with pneumatically operated pistons producing short duration impulsive loads have been reported by Hansen in 1949–1951.[101,102] Similar tests have also been reported in England.[103] A primary purpose of all tests of this type is the determination of the ability of a beam to absorb the energy of impact, and the deformations of the beams thus assume an importance equal to that of the ultimate strength. In this respect, impact tests have shown clearly the importance of ductile modes of failure, either flexural failures by yielding of the reinforcement or shear failures of beams with web reinforcement. The importance of web reinforcement in adding to the toughness of the member has been emphasized by the results of impact tests.

Summary

It seems hardly necessary to state in summary that our knowledge of the ultimate strength of beams subjected to both flexure and shear is hardly commensurate with our knowledge of members subjected to the other types or combinations of loading discussed previously. Not only is our knowledge incomplete, but what is worse from the standpoint of research, we are not yet sure just what additional knowledge is needed. Before research can be undertaken, questions must be asked, and the possibilities in this case are two. We may ask: "What resistance to shear must be provided in a beam in order that final failure will be by flexure and that the maximum possible ultimate strength and ultimate deformation will be developed?" Or we may ask: "What is the ultimate strength of a beam failing primarily due to shear, and how is that strength affected by the properties and proportions of the beam or by the presence of moment?" The direction taken by research is quite different for the two questions, and whether one or the other, or even both, questions should be asked is a matter of opinion regarding design philosophy and is therefore outside the scope of this paper.

MEMBERS SUBJECTED TO COMBINED FLEXURE, AXIAL COMPRESSION AND SHEAR

Members subjected to all three types of stress—flexure, axial compression, and shear—are quite common in reinforced concrete structures. Examples include both columns and beams in building frames, especially if lateral loads are considered, tunnel linings, box culverts, and arches.

Little experimental research has been devoted to this type of member. The only test known to the writer that was planned specifically to study this stress condition was reported by Mörsch in 1929.[104] Only one specimen was tested. It consisted of a prismatic member provided with brackets on

opposite sides at its ends so that load could be applied along a line inclined to the axis of the member and crossing the axis at mid-length. The specimen had no web reinforcement and was designed to fail in shear, which it did.

The only other tests known that may be considered to bear on this problem are those on "knee frames" from rigid frame bridges. Tests on 12 such members were reported by Hayden in 1923,[105] and tests on 32 specimens were reported by Richart and Olson in 1937.[106] The latter program included eight knee frames tested under service loads sustained for 15 to 18 months. These specimens were subsequently tested to failure in short-time tests in which it was found that their ultimate strengths were substantially the same as those for specimens that had not been subjected to sustained loads. Failure in all of these tests was in flexure and with one exception was due to yielding of the reinforcement.

This apparent lack of research on ultimate strength of members subjected to the fairly common combination of flexure, axial compression, and shear is undoubtedly due to the complexity of the problem and the difficulty of its solution in view of our present knowledge, rather than to any lack of appreciation of the importance of this type of member in relation to actual structures. The solution of this problem may indeed be considered the final goal of all research on ultimate strength. The ability to predict the strength of a member subjected to flexure, axial compression, or shear, or any combination of these stresses, when subjected to loads of different time characteristics, will require a more complete and more penetrating knowledge of the behavior of reinforced concrete members than is likely to be obtained for many years to come.

SUMMARY

In the light of the foregoing discussions, the present state of our knowledge regarding the ultimate strength of reinforced concrete members can be summarized. The extent of our knowledge can best be viewed in terms of ultimate strength theories, defined for this purpose as theories capable of predicting the mode of failure and ultimate strength of a given reinforced concrete member subjected to a given type of loading.

In the portion of this paper headed "Scope," five types of stress conditions and five types of loadings have been listed. Taken together, these represent 25 possible conditions for which the ultimate strength of a member need be known. Of these 25 combinations, we have satisfactory ultimate theories, proved by tests, for only two, or possibly three, conditions. For pure flexure or pure axial compression, and for static loads only, the available theories are entirely satisfactory, while for combined flexure and axial compression under static loads, theories are available which require only a relatively small amount of additional experimental checking before they can be accepted finally. However, we do not now possess adequate ultimate theories for members subjected to combinations of flexure and shear, or flexure, compression and shear. Nor do we have theories to predict the ultimate strengths

for any of these types of members for loadings other than static. Although a number of tests have been made in fatigue and under sustained working loads, the conditions for failure under dynamic loading or under sustained high loads have barely been explored.

Taken as a whole, knowledge of ultimate strength in only three cases out of 25 may appear to be a small accomplishment. This is not true, however, since not all of the cases cited are of equal importance, and the work done thus far has necessarily been concentrated on those problems most fundamental in nature and most amenable to solution. The understanding and insight into the behavior of structural members which has been acquired in the solution of these problems will be found invaluable in extending research on ultimate strength into the realms of more complicated states of stress and loadings.

The foregoing discussions and summary may have left the impression that our knowledge of ultimate strengths leaves much to be desired. If so, it should not be inferred that this deficiency affects only our ability to design members by ultimate procedures. It must be remembered, that whether designs are based on working stresses and factors of safety or on ultimate strengths and load factors, both procedures are empirical and derive their validity solely from tests. Any lack of sufficient test data is as much a deficiency in one method of design as in the other. Design procedures and design philosophies may change, but safe and economical design is based on the unchanging need for knowledge of the properties and behavior of reinforced concrete members, and this can be obtained only from tests.

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Tests for Ultimate Loads

By G. C. ERNST*

Inasmuch as the foundation of ultimate load design consists of the existing information from ultimate load tests, it would seem that the excellent educational effort exemplified by this symposium should also include an explanation of the various test methods by which ultimate load values for specific stress conditions have been obtained. Although it is true that many have a poor recollection of the ultimate load formulas for beams proposed by the veteran savants of reinforced concrete at the turn of the century, it is equally true that an even poorer understanding maintains as to the test methods by which various static stress conditions are developed for ultimate load tests. Rectangular and tee-beams, short columns, bond tests, and the 6 x 12-in. cylinder are traditional; and many are acquainted, no doubt, with the continuous beam; rigid frame, knee frame, and arch tests that have been made in recent years.

However, the isolation and recombination of loading arrangements to produce specific stress conditions touched upon briefly by Professor Siess in his summary of 65 years of reinforced concrete testing, have long posed problems that have prevented a completely satisfactory determination of the static ultimate load for all design requirements. Professor Siess reviewed five types of fundamental stress conditions that have been investigated, some in rather limited number. Certainly, for a satisfactory understanding of the action at static ultimate loads all possible combinations of the three basic stress conditions (flexural, direct, and shear) must be studied.

*Member American Concrete Institute, Professor and Chairman, Dept. of Civil Engineering, University of Nebraska, Lincoln.

Fig. 1 illustrates the approximate present status of ultimate load testing, and also serves to demonstrate some of the difficulties encountered in reproducing a desired stress condition with testing equipment currently used. For example, taking combined bending (flexure) and direct stress, commonly termed "eccentric loading" for columns, four different test procedures have been employed depending upon the type of structural component or member tested. These four test procedures are diagrammatically shown in the lower left hand portion of Fig. 1. Insofar as columns are concerned, however, it has been traditional to test with a constant eccentricity, e, the axial load increasing with the bending moment up to failure. In 1938, F. G. Thomas[1] reported sixteen tests in which the ultimate strength was determined for columns subjected to continuously changing eccentricities by means of the first test method shown in Fig. 1. The writer knows of no further tests of this character for columns, although R. W. Kluge (as well as the writer) produced a more clearly defined combination of direct stress and bending for reinforced concrete articulations (hinges) following the third method in the lower left hand enclosure of Fig. 1. In fact, in the case of the writer's tests on Considère hinges, the section under direct stress and bending was an extremely short spiral column, and performed as such. The results reported from both groups of these tests[2,3] are consistent, in the writer's opinion, in showing a need for further study on columns under variable eccentricities. Of the two possible conditions, (1) increasing thrust at constant eccentricity, or (2) increasing angular rotation at fixed thrust, the writer considers the second to be of wider application in design,[4] but far more difficult to reproduce and study in the laboratory. This is also true for combined thrust and shear, or combined bending, thrust, and shear.[3] Our ultimate load test information is significantly meager on such combinations. Furthermore, with the growing use of extremely thin sections of reinforced concrete, a better knowledge is needed of instability forms of failure. In thin-shelled construction, the writer knows of only six tests that were competently set up and controlled.

In view of such deficiencies, it would not seem desirable to attempt a major change to ultimate load design at this time, European codes to the contrary notwithstanding. The door should never be closed to either philosophy, since a few remnants of the so-called elastic theory will be needed to clothe the bare spots of the ultimate load theory, and vice versa.

In closing, the writer wishes to emphasize that he has been an advocate of certain phases of ultimate load design, for both reinforced concrete and steel, for many years; but he sees little merit in a complete conversion to it at this time. Our need is not that of a "code change," but rather that of a more carefully cultivated "intellectual curiosity" combined with an "insatiable desire for the truth."

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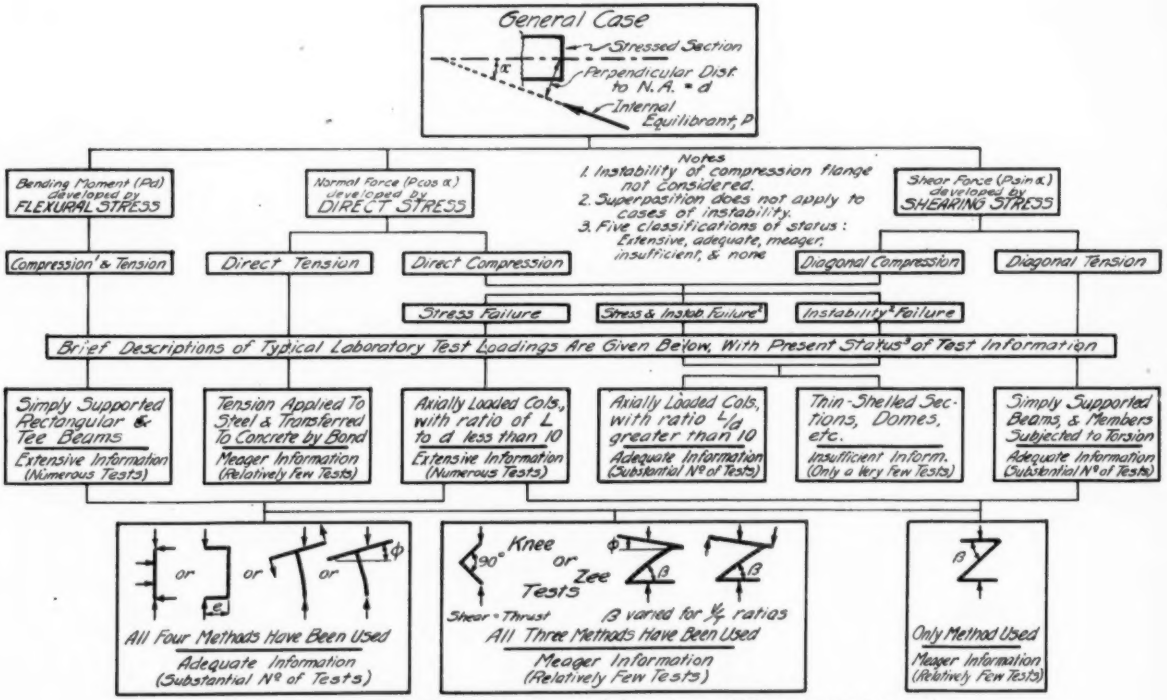
General Case
Stressed Section
Perpendicular Dist. to N.A. = d
Internal Equilibrant, P
Notes
1. Instability of compression flange not considered.
2. Superposition does not apply to cases of instability.
3. Five classifications of status: Extensive, adequate, meager, insufficient, & none
Bending Moment (Pd) developed by FLEXURAL STRESS
Normal Force (P cos α) developed by DIRECT STRESS
Shear Force (P sin α) developed by SHEARING STRESS
Compression & Tension
Direct Tension
Direct Compression
Diagonal Compression
Diagonal Tension
Stress Failure
Stress & Instab. Failure
Instability Failure
Brief Descriptions of Typical Laboratory Test Loadings Are Given Below, With Present Status of Test Information
Simply Supported Rectangular & Tee Beams
Extensive Information (Numerous Tests)
Tension Applied To Steel & Transferred To Concrete by Bond
Meager Information (Relatively Few Tests)
Axially Loaded Cols, with ratio of L to d less than 10
Extensive Information (Numerous Tests)
Axially Loaded Cols, with ratio L/d greater than 10
Adequate Information (Substantial No of Tests)
Thin-Shelled Sections, Domes, etc.
Insufficient Inform. (Only a Very Few Tests)
Simply Supported Beams, & Members Subjected to Torsion
Adequate Information (Substantial No of Tests)
or
or
or
All Four Methods Have Been Used
Adequate Information
(Substantial No of Tests)
90°
Knee
or
Zee
Tests
Shear = Thrust
β varied for L/d ratios
All Three Methods Have Been Used
Meager Information
(Relatively Few Tests)
Only Method Used
Meager Information
(Relatively Few Tests)

Note: Other combinations in fundamental form have not been reported to date.

Fig. 1—Summary of fundamental types of static ultimate load tests on reinforced concrete members

2. *Bulletin* 322, University of Illinois Engineering Experiment Station.

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Discussion

By A. L. PARME*

In his excellent paper, Professor Siess has given a well organized panoramic view of the tremendous number of tests which serve as the basis for the ultimate strength theory of design. Without a doubt it can be safely said that no other design theory has been so amply verified. The reason for this abundance of supporting evidence is quite obvious because the ultimate strength theory of design represents basically only a more realistic interpretation of the same data which serve as the foundation for the present-day design procedures. In this sense, ultimate strength theories, by returning the proportioning of sections back to well established facts, bring the design of concrete structures abreast with the research and developments in concrete technology during the past forty years.

A complete appraisal of the soundness of ultimate load design cannot be undertaken without considering the vast fund of knowledge which has been accumulated since the straight-line theory was established as a design procedure. To list all of the contributions to knowledge of concrete since the turn of the century is considerably beyond the scope of this short discussion. However, since uniformity in the strength of concrete is of major importance in ultimate load design, two factors are worth mentioning. First, when the straight-line theory was adopted by the First Joint Committee, the strength of concrete was considered to be controlled by the proportions of the mix. It was not until nine years later that Duff Abrams enunciated his famous water-cement ratio principle for the design of concrete mixes. The marked improvement in concrete making which this single contribution has made is too apparent to need discussion.

The second and equally pertinent factor is that because of extensive educational work over a long period to improve the quality of concrete, uniformity of concrete strength is being obtained on the average well controlled jobs. Thus for example, in a study of 13,000 field tests taken from 60 jobs,[1] only 5 percent of the test cylinders showed strengths below 85 percent of the strengths used in design. This uniformity is likewise attested by findings of the Bureau of Reclamation.[2] In small structures where the variation in the strength of the test cylinders was greatest, it was only 12.6 percent.

*Member American Concrete Institute, Structural Engineer, Structural and Railway Bureau, Portland Cement Assn., Chicago, Ill.

Moreover, H. L. Kennedy reported[3] that an extensive survey has shown that the average compressive strength of specimens core drilled from hardened concrete was 23 percent greater than the average strength for the test cylinders. Thus a designer has reasonable assurance on well controlled jobs, because the strength of concrete in the structure equals or exceeds the specified design strength, that the strength of any member whether its resistance is determined by tension or compression will be equal to the calculated value.

The validity of ultimate strength theories rests on the assumptions that as the ultimate compressive strength of a section is approached, the stresses are no longer proportional to the strains. Repeated direct verification of this has been demonstrated by tests on columns and restrained and unrestrained cylinders and indirectly in beam tests. More recently a direct confirmation of this hypothesis for members subject to flexural forces has been achieved by the Bureau of Reclamation. As a result of the development of a small pressure cell they were able to make direct measurements of the stresses occurring in a concrete beam. The results of a test are shown in Fig. 2. It can be seen that although the strains are linear up to failure, the distribution of compressive stresses is curvilinear. Another significant fact revealed is that the measured ultimate stress is equal to the cylinder strength. The breaking load calculated by two ultimate strength procedures agrees

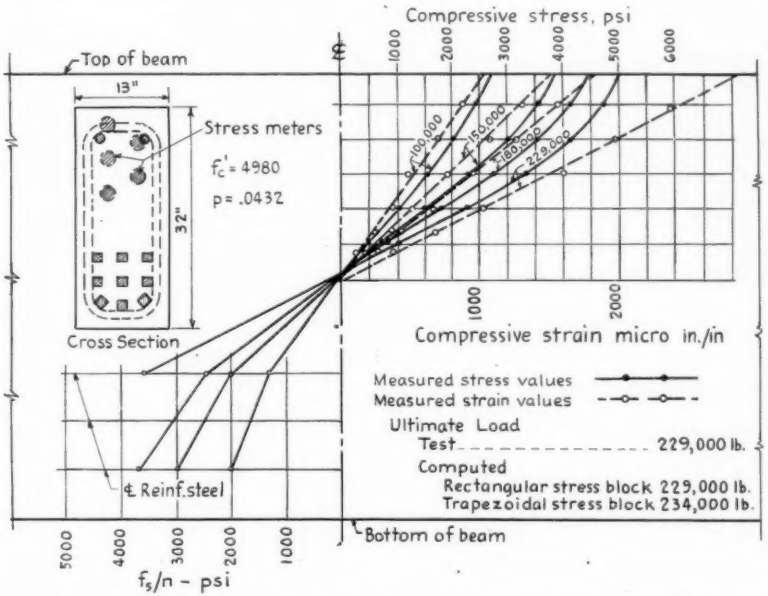

Fig. 2—Result of Bureau of Reclamation test on reinforced concrete beam with Monfore stress meters and SR-4 gages

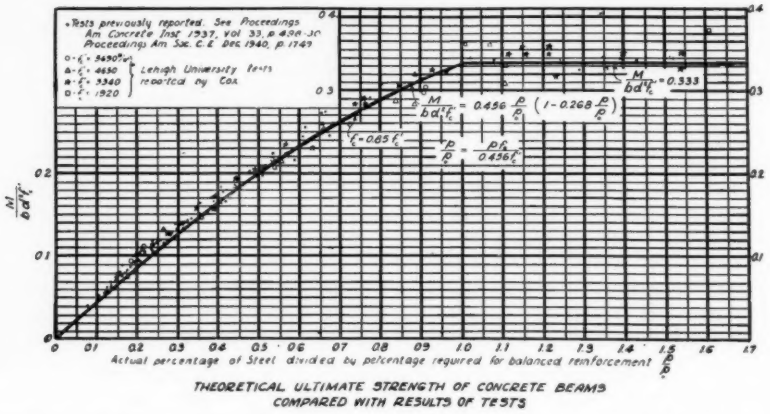
Tests previously reported. See Proceedings Am. Concrete Inst. 1937, vol. 33, p. 484 or Proceedings Am. Soc. C. E. Dec. 1940, p. 1749
Lehigh University tests reported by Cox
$\frac{M}{bd^2f_c}$
$\frac{M}{bd^2f_c} = 0.456\frac{p}{p_1}\left(1-0.268\frac{p}{p_1}\right)$
$\frac{M}{bd^2f_c} = 0.333$
$f_c = 0.65f'_c$
$\frac{p}{p_1} = \frac{pf_c}{0.456f'_c}$
Actual percentage of Steel divided by percentage required for balanced reinforcement $\frac{p}{p_1}$
THEORETICAL ULTIMATE STRENGTH OF CONCRETE BEAMS COMPARED WITH RESULTS OF TESTS

Fig. 3—Theoretical ultimate strength of concrete beams compared with results of tests

closely with the test value. Similar verification is also being obtained by photo-elastic studies on concrete beams at Ohio State University.

Fig. 3, taken from Whitney's article in the *Journal* of the Boston Society of Civil Engineers, is typical of the excellent agreement obtained between tests and values calculated by ultinate strength theories for members failing in flexure. A significant point generally overlooked is that the test beams are subject to concentrated loads considerably in excess of the dead weight, and therefore near maximum shear and maximum moment values occur at the same place. The close agreement shown is a strong indication that the flexural strength is independent of the shear stresses. This fact is likewise confirmed by studies made on tests of rigid knee frames.

In his paper Professor Siess has indicated with some emphasis the need for more sustained, repeated and fatigue tests. The necessity for such data is dependent primarily on the overload factors selected in ultimate load design. The smaller the range between service loads and ultimate loads, the more important these tests become. However, since the overload factors generally contemplated give an over-all factor of safety comparable to those prevailing now, there is some question whether an immediate need for such tests exists. The principal value of such tests will be in enabling a reduction in the overload factors.

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Discussion of a paper by C. P. Siess:

Review of Research on Ultimate Strength of Reinforced Concrete Members*

By H. J. COWAN and S. ARMSTRONG, and AUTHOR

By H. J. COWAN and S. ARMSTRONG†

The writers were interested to note the remarks of Professor Siess on the paucity of information on the strength of reinforced concrete in torsion and under the action of combined stresses. There is, in fact, considerable literature on the subject of pure torsion, although much of it does not appear to be widely known.[1-11]

There are three investigations on record relating to combined bending and torsion. Fisher[12] made tests on plain concrete beams, Nylander[13] on beams with longitudinal reinforcement, and the writers[14,15] on beams with longitudinal reinforcement and vertical stirrups.

The results show that torsional strength of reinforced concrete depends essentially on the properties of the concrete. Although torsional strength can be augmented by addition of shear reinforcement, character of the failure is not thereby altered. The tensile stresses in the steel combine with the compressive stresses in the concrete to provide the torsional resistance moment. However, whereas in bending, the steel in tension and the concrete in compression are on opposite faces of the beam, they are not physically separated in a member subject to torsion. Consequently straining of the concrete produces cleavage cracks in the concrete resisting compression parallel to the direction of the principal compressive stresses. These cracks eventually lead to failure.

Before the beam fails, a redistribution of stresses takes place due to the inelastic deformation of the concrete. The inelastic deformation of the concrete does not immediately produce plastic straining in the shear reinforcement, since the yield strain of steel is at least twice as great as the ultimate tensile strain of concrete. The steel therefore remains elastic almost up to the point of failure. In some cases the beam may fail before the steel yields.

Since the resistance of reinforced concrete beams to torsion depends on the tensile strength of the concrete, it is not possible to gain an increase in strength comparable with that resulting from the addition of tension reinforcement

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†Lecturers in Civil Engineering, University of Sheffield, Sheffield, England.

to a plain concrete beam subject to bending. On the other hand, the insertion of even a small quantity of shear reinforcement turns the abrupt torsion fracture of plain concrete into a gradual failure preceded by considerable cracking and deformation.

In reinforced concrete subject to combined bending and torsion there is a transition from primary bending to primary torsion failure. The transition occurs when the torsional shear stress on the compression face of the beam becomes sufficiently large to produce the limiting principal tension corresponding to primary torsion failure. The compression due to bending reduces the principal tension due to torsion, and the addition of a small amount of bending therefore results in some increase in torsional strength. It is found that the addition of a small amount of torsion does not materially reduce bending strength. It therefore appears reasonable to design reinforced concrete sections subject to combined bending and torsion to resist independently the torsional and bending moments, without any reduction in the ultimate strength of the concrete in either compression due to flexure or diagonal tension due to torsion.

The mathematical treatment of the arguments set out above will be found in two papers now awaiting publication.[16,17]

The writers are also currently investigating the problem of prestressed concrete in combined bending and torsion. This may be expected to yield information on the ultimate strength of members subject to combined flexure, axial compression, and shear. Preliminary results were reported in the discussion at the Symposium on Prestressed Concrete Statically Indeterminate Structures held by the Cement and Concrete Assn. in London in September, 1951.

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AUTHOR'S CLOSURE

This paper was obviously not comprehensive, nor was it intended to be. Space limitations, for example, precluded any discussion of tests on hinges or on reinforced concrete in pure tension. Consequently, the contributions to these subjects by Professor Ernst* and by Dr. Cowan and Mr. Armstrong, respectively, represent a valuable addition to the original paper.

Further limitation on the scope of the paper was imposed by the incompleteness of the author's knowledge of the many tests that have been made. He is especially grateful, therefore, to Dr. Cowan and Mr. Armstrong for their discussion and references relating to tests of reinforced concrete members subjected to combined bending and torsion. It is gratifying to learn that this important subject is now receiving attention.

Mr. Parme† sees in the results of certain tests "a strong indication that the flexural strength of a reinforced concrete beam is independent of the shear stresses." Although this statement may be justified in the case of simple beams loaded at the third-points, the important question is whether it applies also to the more commonly encountered case of restrained beams subjected to uniform loading.

Mr. Parme mentions also that the lack of knowledge gained from tests under sustained or repeated loads need not prevent the use of ultimate load design procedures. This, of course, is true, but only if the structure designed on an ultimate load basis is essentially similar to one that would be designed by present procedures. It is important to realize that if this condition is not satisfied, design for ultimate loads may represent a venture into a region in which our knowledge is far from complete.

In a letter to the Editor published in the October, 1952, JOURNAL,‡ F. A.

*ACI JOURNAL, June 1952, *Proc.* V. 48, p. 859.
†ACI JOURNAL, June 1952, *Proc.* V. 48, p. 862.
‡"Comments on Symposium on Ultimate Load Design (LR 49-9)," Letters from Readers, ACI JOURNAL, Oct. 1952, *Proc.* V. 49, p. 159.

Blakey cites the unbalance in research on ultimate strength as a reason for "the reluctance of design engineers in general to accept ultimate strength methods . . . " In the author's opinion, this is not a valid reason, since any deficiencies in test data or research apply to current methods of proportioning as well as to those based on ultimate strengths. However, this last statement applies only if a distinction is made between the use of ultimate loads and strengths to determine the size and proportions of a member, and the use of true ultimate design theories such as "limit design." It has not always been clear which of these philosophies was being advocated.

Practical Design at Ultimate Loads*

By R. C. REESE†

SYNOPSIS

After a few comments on the fundamental philosophy of ultimate load design, a short discussion of the formulas used for practical design (rectangular stress prism) is given. Designs are carried out for slabs and beams and for columns with and without bending by the conventional and ultimate load design methods. Some comparisons are made of the results as regards amount of space occupied, time sagging of members and cost of construction. Remarks are offered on the appropriateness of using elastic frame analysis, plastic theories and the various ultimate stress prisms.

INTRODUCTION

It is impossible to illustrate structural design without considerable recourse to mathematics. If the computations are slighted, the practicing designer is not helped. Therefore, a typical panel is designed by the conventional method and by ultimate load design. An attempt is made to obtain intelligible working results without presenting too many detailed figures.

The present, uniformly-varying, straight-line assumption of stress distribution within the elastic range cannot be extended to predict failure of a test beam with any degree of accuracy. Conversely, it is possible to analyze the reported results of all beams tested to destruction and to develop formulas, considering all recognized variables, that will agree quite closely with the test results. If the action of reinforced members in a structure cast monolithically agrees reasonably well with the behavior of individual, simply-supported, free-standing test members, it is possible to formulate a design method. If the designer can predict failure with a high degree of precision, he can work closer to this limit with relatively lower overload factors and with different values for the dead loads, which are readily computed, than for the live loads, which are assumed as representative of the proposed occupancy. Considerably more latitude in judgment is then possible to meet unusual conditions, but care is required to prevent excessive deflections through unduly shallow members. Further development of methods for shear, bond, and diagonal tension are needed to go along with the well-developed flexural methods.

ULTIMATE LOAD METHODS

Column design

Since 1941, columns have been designed by ultimate load methods. Prior

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†Member American Concrete Institute, Consulting Engineer, Toledo, Ohio.

to that, codes suggested that columns be designed by the elastic theory, assuming that, because of equal deformations, the concrete and vertical bars would divide the total load according to their relative moduli of elasticity. As a result of tests reported in 1933,[1] a method was developed considering the transfer of load to the bars up to their yield point because of the plastic action of the concrete. Accordingly, the ultimate strength of a spirally reinforced column may be computed from the summation of the capacity of the concrete at 85 percent of cylinder strength and the capacity of the vertical bars at yield-point stress. This method favored higher percentages of reinforcement and high-yield-point bars in order to reduce the size of columns and increase usable floor space. Actual working stresses are well within the elastic range, but the ability to predict failure and the recognition of the transfer of load allows the application of a proper overload factor at the yield point in lieu of using working loads at working stresses. There is, then, precedent for ultimate load design.

Safety factor

The advantages of ultimate load design of flexural members include a more uniform factor of safety, which means that some multiple of the proposed loads, *i.e.*, overload factor, would cause failure of all members at about the same time. If the failure point is well defined, it is practical to work much closer to this point and adjust overload factors to correspond. Also it is somewhat more consistent to use the same method of attack on flexural members as on columns; particularly is it desirable to have a uniformity of method when it comes to dealing with members undergoing combined axial and bending stress, as in the case of eccentrically loaded columns, arch ribs, and the like. There is a pronounced need for a more precise method for long-span structures, where dead weight is a large factor in design.

Plastic action

Consideration must be given to the meaning of the word *plastic*, which is used indiscriminately to mean either a time-yielding (creep) of the concrete under continued load or simply a loading of such intensity as will produce stresses above the elastic range of the concrete and up to the ultimate load. One is a long-time creep under sustained load; and the other, an immediate excursion into a stress intensity above the elastic limit, *i.e.*, plastic action.

Stress block

Experiments performed above the elastic limit show that Bernoulli's hypothesis of a section, plane before bending, remaining plane after bending, is reliable. Hence, strains vary as the distance from the neutral axis, and the prism of compressive stress approximates the stress-strain curve, if we assume that the strain characteristics of individual fibers, in flexural members, are not materially different from axially-stressed test specimens. Many proposals have been made for the shape of the curve. Passing over several miscellaneous suggestions, the main possibilities include a fifth-power parabola (Lyse), a rectangle (Whitney), a quadratic parabola and inclined right line (Stüssi,

Hognestad), and a trapezoid, where one inclined line represents the elastic range and a line at another inclination, the plastic range (Jensen). All predict failure rather accurately, having been deduced from test results, and all require the insertion of constants derived from experiment. There is not a great deal to choose between them, except that the formulas based upon a rectangle are slightly less complicated.

COMPARISON OF CONVENTIONAL AND ULTIMATE LOAD METHODS

A comparative study will now be made of designs by the present (working-load, triangular stress prism) method and by the proposed ultimate load design method (rectangular stress prism) for a beam-and-girder building with columns at 20-ft centers each way, beams at 10-ft centers one way and girders at 20-ft centers the other (Fig. 1), for a live load of 130 psf to include any partitions and finishes, using 3000 psi concrete and intermediate grade reinforcing bars with a minimum yield point of 40,000 psi.

Slab design—Conventional methods

In Example 1, a slab thickness of 3½ in., weighing 44 psf, produces a total load of 174 psf. The effective depth is 2½ in.; the span, 9 ft 4 in. Using ACI Building Code coefficients, the positive and negative moments are computed. $R = M/bd^2$ is figured for negative moment as 220, or slightly less than the 235 of balanced reinforcement, but not enough smaller to suggest using a thinner slab. Negative and positive steel areas are worked out, and these areas are easily made. The cost of this slab was figured assuming that form costs, finishes, and so forth would remain constant. Concrete was taken at $13.50 per cu yd in place (4⅙ cents per fbm) and reinforcing steel at 11 cents per pound in place. This estimated cost per square foot deals only with the slab itself with no consideration, for the time being, in saving of dead weight or total height of structure, or column and footing sizes.

Example 1—3½-in. slab, span 9 ft 4 in., $d = 3½ - ¾ - ¼ = 2½$ in.

Live load	= 130
Est. 3½-in. slab	= 44
	174 psf

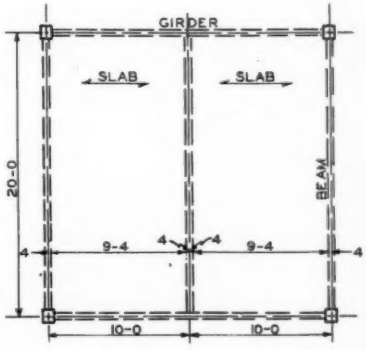

Fig. 1—Slab supported by beam-and-girder framing.

$$+M = \frac{wL^2}{16} = \frac{174 \times (9.33)^2 \times 12}{16} = 11{,}360 \text{ in.-lb}$$

$$-M = \frac{wL^2}{11} = \frac{174 \times (9.33)^2 \times 12}{11} = 16{,}510 \text{ in.-lb}$$

$$R = \frac{M}{bd^2} = \frac{16{,}510}{12 \times (2\tfrac{1}{2})^2} = 220 < 235$$

$$-A_s = \frac{M}{f_s\, jd} = \frac{16{,}510}{20{,}000 \times 0.869 \times 2\tfrac{1}{2}} = 0.380 \text{ sq in.}$$

$$+A_s = \frac{M}{f_s\, jd} = \frac{11{,}360}{20{,}000 \times 0.886 \times 2\tfrac{1}{2}} = 0.256 \text{ sq in.}$$

Cost: $3\frac{1}{2}$-in. concrete @ $4\frac{1}{6}$ cents = 14.58
Positive steel — 0.256
Extra — 0.4×0.124 = 0.050
0.306×3.4 @ 11 cents = 11.45
26.03 cents per sq ft

For comparative purposes, alternative slabs 4 in. and $4\frac{1}{2}$ in. thick were designed and costed (Example 2), resulting in very slightly greater costs per square foot at the unit prices here assumed, showing that balanced reinforcement is economical in this case.

Example 2

4-in. slab, $d = 3$ in.
Live load = 130
4-in. slab = 50
180 psf

$$+A_s = \frac{180 \times (9.33)^2 \times 12}{16 \times 20{,}000 \times 0.90 \times 3} = 0.217 \text{ sq in.}$$

$$-A_s = \frac{180 \times (9.33)^2 \times 12}{11 \times 20{,}000 \times 0.886 \times 3} = 0.322 \text{ sq in.}$$

Cost: 4-in. concrete @ $4\frac{1}{6}$ cents = 16.67
Positive steel— 0.217
Extra—0.4×0.105 = 0.042
$0.259 \times 3.4 \times 11$ cents = 9.70
26.37 cents per sq ft

$4\frac{1}{2}$-in. slab, $d = 3\frac{1}{2}$ in.
Live load = 130
$4\frac{1}{2}$-in. slab = 56
186 psf

$$+A_s = \frac{186 \times (9.33)^2 \times 12}{16 \times 20{,}000 \times 0.915 \times 3\tfrac{1}{2}} = 0.190$$

$$-A_s = \frac{186 \times (9.33)^2 \times 12}{11 \times 20{,}000 \times 0.898 \times 3\tfrac{1}{2}} = 0.281$$

Cost: $4\frac{1}{2}$-in. concrete @ $4\frac{1}{6}$ cents = 18.75
Positive steel— 0.190
Extra—0.4×0.091 = 0.036
$0.226 \times 3.4 \times 11$ cents = 8.46
27.21 cents per sq ft

Slab design—Ultimate load method

In Example 3, the same problem is solved by ultimate load design, using a 2¾-in. thick slab. Since we are now designing to ultimate capacity, the live load is multiplied by a factor 2.4 and the dead load by 1.2. Such factors must allow adequately for many things; for the assumptions made by the designer, for all inaccuracies in field work, for any departure of the properties of the materials from those assumed, for construction loads, for possible changes in occupancy during the life of the structure, for changes in properties of the materials with aging, and for loads that are sometimes disregarded, such as vibration and possibly earthquake.

The bending moments are computed using the factors in ACI 318-51. These factors are the result of elastic analyses assuming prismatic, homogeneous beams, continuous over a number of supports, the ratio of span lengths limited to the larger being not more than 120 percent of the smaller and the ratio of live to dead loads being limited to three. The appropriateness of applying elastic frame analysis to stresses in the ultimate range can be questioned and has been. It is possible at any joint, knowing the make-up of an under-reinforced section, to determine the negative moment corresponding to yield-point stress in the steel and to consider the joint as a plastic hinge holding such moment and no more. Additional load would then be transferred to the zone of positive moment. With experience, such plastic hinges might simplify the determination of bending moments. There would be considerable deformation, probably some cracking of the concrete, but only at ultimate loads involving the overload factor. So, for the immediate problem, the same moment factors will be used as for the conventional design.

For balanced reinforcement, the computations are quite simple, giving an effective depth of 1.67 in., agreeing quite closely with the 2¾-in. total depth assumed, and a required steel area of 0.686 sq in. per ft, which can be provided only with a congested arrangement of bars. For positive moment, the section will obviously be under-reinforced. The computations are a little more involved, requiring the determination of the height of the rectangular stress prism from which the steel area is readily obtained.

A cost comparison at the same unit prices as before shows this slab somewhat more expensive than that by the conventional method. Since the cost of the structural framework may be as little as 20 percent of the total cost of the structure, it may be found that the thinner slab would save enough in the height of building and the resulting cost of exterior walls, partitions, and pipe stacks to justify its use.

This slab is considerably thinner than the 1928 ACI Building Code limitation of $L/t = 32$ which would be just under 3½ in. The thinner slab may deflect with time and sag a bit, but Swedish engineers have been building in that fashion, saying other materials sag and a small deflection is not too harmful. If exceedingly thin slabs come into vogue, a study of acceptable sag ratios for different occupancies may be desirable, the old reliable $L/360$ used so

long for plastered ceilings not being applicable to this condition. However, exceedingly thin slabs may not be an economical solution.

Example 3—2¾-in. slab, span 9 ft 4 in.

Live load—2.4 × 130 = 312
Est. 2¾-in. slab—1.2 × 34 = 41
353 psf

$$+M = \frac{wL^2}{16} = \frac{353 \times (9.33)^2 \times 12}{16} = 23{,}050 \text{ in.-lb}$$

$$-M = \frac{wL^2}{11} = \frac{353 \times (9.33)^2 \times 12}{11} = 33{,}550 \text{ in.-lb}$$

For balanced reinforcement

$$M = f_c' \frac{bd^2}{3} = 33{,}550 \text{ in.-lb}, \quad d = \sqrt{\frac{33{,}550}{1000 \times 12}} = 1.67 \text{ in.}$$

Slab = 1.67 + 0.75 + 0.25 = 2.67 in. Use 2¾ in.

$$-A_s = 0.456 \frac{f_c' bd}{f_y} = \frac{0.456 \times 3000 \times 12 \times 1.67}{40{,}000} = 0.686 \text{ sq in.}$$

Positive moment (under-reinforced)

$$\frac{a}{d} = 1 - \sqrt{\frac{1 - 2.35M}{f_c' bd^2}} = 1 - \sqrt{1 - \frac{2.35 \times 23{,}050}{3000 \times 12 \times (1.75)^2}} = 0.286$$

$$a = 0.286 \times 1.75 = 0.50$$

$$+A_s = \frac{M}{f_y\left(d - \frac{a}{2}\right)} = \frac{23{,}050}{40{,}000\ (1.75 - 0.25)} = 0.384 \text{ sq in.}$$

Cost: 2¾-in. concrete @ 4⅙ cents per sq ft = 11.45
Positive steel— 0.384
Extra—0.4 × 0.302 = 0.121
0.505 × 3.4 × 11 cents = 18.85
30.30 cents per sq ft

In Example 4, 3½- and 4-in. slabs are designed in a similar manner as alternatives. When priced with the same units, costs are less than that of the thinner slab and comparable with the same slab thicknesses designed by the conventional method. This merely indicates that the overload factors used correspond reasonably well with the safety factors now in vogue.

Example 4

3½-in. slab, $d = 2\frac{1}{2}$ in.

Live load—2.4 × 130 = 312
3½-in. slab—1.2 × 44 = 53
365 psf

$$+M = \frac{wL^2}{16} = \frac{365 \times (9.33)^2 \times 12}{16} = 23{,}850 \text{ in.-lb}$$

$$-M = \frac{wL^2}{11} = \frac{365 \times (9.33)^2 \times 12}{11} = 34{,}700 \text{ in.-lb}$$

$$+\frac{a}{d} = 1 - \sqrt{1 - \frac{2.35 \times 23{,}850}{3000 \times 12 \times (2.5)^2}} = 0.133$$

$$+ A_s = \frac{23{,}850}{40{,}000 \times 2.5 \times 0.933} = 0.255 \text{ sq in.}$$

$$- \frac{a}{d} = 1 - \sqrt{1 - \frac{2.35 \times 34{,}700}{3000 \times 12 \times (2.5)^2}} = 0.202$$

$$- A_s = \frac{34{,}700}{4000 \times 2.5 \times 0.899} = 0.385 \text{ sq in.}$$

Cost: $3\frac{1}{2}$-in. concrete @ $4\frac{1}{6}$ cents = 14.58
Positive steel— 0.255
Extra—0.4 × 0.130 = 0.052
0.307 × 3.4 × 11 cents = 11.50
26.08 cents per sq ft

4-in. slab, $d = 3$ in.
Live load—2.4 × 130 = 312
4-in. slab—1.2 × 50 = 60
372 psf

$+ M = 24{,}300$; $- M = 35{,}400$

$$+ \frac{a}{d} = 1 - \sqrt{1 - \frac{2.35 \times 24{,}300}{3000 \times 12 \times (3)^2}} = 0.091$$

$$+ A_s = \frac{24{,}300}{40{,}000 \times 3 \times 0.954} = 0.212$$

$$- \frac{a}{d} = 1 - \sqrt{1 - \frac{2.35 \times 35{,}400}{3000 \times 12 \times (3)^2}} = 0.138$$

$$- A_s = \frac{35{,}400}{40{,}000 \times 3 \times 0.931} = 0.317$$

Cost: 4-in. concrete @ $4\frac{1}{6}$ cents = 16.67
Positive steel— = 0.212
Extra—0.4 × 0.105 = 0.042
0.254 × 3.4 × 11 cents = 9.50
26.17 cents per sq ft

These first computations illustrate the conventional and ultimate load designs, giving some idea of the amount of effort required on each. It is only fair to say that, since the ultimate load theory is the newer, more formulas are given for the convenience of the reader. With the overload factors here assumed, the ultimate load theory allows shallower sections, providing codes are revised to require deflection computations and eliminate arbitrary depth-to-span ratios. Ultimate load designs give a more realistic approach to the stress problem and put the designer closer to the facts upon which his judgment is based. This set of computations illustrates only the application of the ultimate load method to simple rectangular beams.

Comparison of long-span slab design methods

Before proceeding to the beams of this typical building, it is interesting to compare conventional and ultimate load design for a long-span slab, where dead weight becomes a greater factor. In Example 5, a solid slab is designed by both methods for a single span of 30 ft, with a live load of 100 psf. The difference in thickness is readily apparent. The cost per square foot of slab

alone is just about a stand-off; but the saving in columns, footings, height of building and other factors can be considerable.

Example 5—Long-span solid slab

Conventional method—$L = 30$ ft 0 in., $t = 10\frac{1}{2}$ in., $d = 9.0$ in.

Live load = 100
Dead load = 131
231 psf

$$+M = \frac{wL^2}{16} = 156{,}000 \text{ in.-lb}$$

$$-M = \frac{wL^2}{11} = 228{,}000 \text{ in.-lb}$$

$-R = 234 < 235$, $-A_s = 1.45$ sq in., $+A_s = 0.99$ sq in.

Cost: $10\frac{1}{2}$-in. concrete @ $4\frac{1}{6}$ cents = 43.8
Steel—$[0.99 + (0.4 \times 0.46)] \times 3.4 \times 11$ cents = 44.0
87.8 cents per sq ft

Ultimate load method—$L = 30$ ft 0 in., $t = 6\frac{1}{2}$ in., $d = 5\frac{1}{4}$ in.

Live load—2.4×100 = 240
Dead load—$1.2 \times 81\frac{1}{4}$ = 98
338 psf

$$+M = \frac{wL^2}{16} = 228{,}000 \text{ in.-lb}$$

$$-M = -\frac{wL^2}{11} = 332{,}000 \text{ in.-lb}$$

$$d = \sqrt{\frac{332{,}000}{1000 \times 12}} = 5.25$$

$$-A_s = 0.456\frac{f_c' bd}{f_y} = 2.16 \text{ sq in.}$$

$$\frac{a}{d} = 1 - \sqrt{1 - \frac{2.35 \times 228{,}000}{3000 \times 12 \times (5.25)^2}} = 0.322; \quad a = 1.69$$

$$+A_s = \frac{228{,}000}{40{,}000\,(5.25 - 0.85)} = 1.29 \text{ sq in.}$$

Cost: $6\frac{1}{2}$-in. concrete @ $4\frac{1}{6}$ cents = 27.1
Steel—$[1.29 + (0.4 \times 0.87)]\,3.4 \times 11$ cents = 61.3
88.4 cents per sq ft

Floor beam—Conventional method

Returning to the typical building, Examples 6 and 7 design a typical floor beam for the same panel by each method. The conventional method (Example 6) parallels the slab just completed so closely that not much explanation is required down to the computation of the necessary compressive reinforcement at the support, where the theoretical amount is reduced 50 percent in accordance with the provision for plastic action discussed in ACI 318-51, 706(b), which, again, is an attempt to recognize plastic action while still conforming to the elastic theory. The design is costed at the units previously assumed.

Example 6—Beam, $L = 19$ ft 0 in.,

10 ft @ 174 lb = 1740
8 x 15-in. stem = 120
1860 psf

$$V = \frac{wL}{2} = \frac{1860 \times 19}{2} = 17{,}680 \text{ lb}$$

$$+M = \frac{wL^2}{16} = \frac{1860 \times (19)^2 \times 12}{16} = 504{,}000 \text{ in.-lb}$$

$$-M = \frac{wL^2}{11} = \frac{1860 \times (19)^2 \times 12}{11} = 733{,}000 \text{ in.-lb}$$

Try 8 x 18 in., $d = 15\frac{1}{2}$ in., $d' = 2\frac{1}{2}$ in., $\frac{d'}{d} = 0.161$

$$-R = \frac{M}{bd^2} = \frac{733{,}000}{8 \times (15\frac{1}{2})^2} = 382 > 235$$

$p = 0.0225$ $\quad -A_s = 2.79$ sq in.

$$p' = \frac{0.024}{2} = 0.012 \quad -A_s' = 1.49 \text{ sq in. (ACI 706-b)}$$

$$v = \frac{V}{bjd} = \frac{17{,}680}{8 \times \frac{7}{8} \times 15\frac{1}{2}} = 163 > 90 < 360$$

$$v_{@} = \frac{1300 \times 19}{8 \times 8 \times \frac{7}{8} \times 15\frac{1}{2}} = 29 \text{ psi}$$

$$a = \frac{19 \times 12 \times (163 - 90)}{2\,(163 - 29)} = 62$$

$$A_v = \frac{62 \times 8 \times (163 - 90)}{2 \times 20{,}000} = 0.91 \text{ sq in.}$$

9–# 3 stirrups @ 3 in., 8 @ 6½ in. (ACI 806 (a))

$$\frac{t}{d} = \frac{3\frac{1}{2}}{15\frac{1}{2}} = 0.226 \qquad R_{max} = 198$$

$$+b = \frac{M}{Rd^2} = \frac{504{,}000}{198 \times (15\frac{1}{2})^2} = 10.6 < \frac{19 \times 12}{4} < 10 \times 12 < 8 + 16 \times 3\frac{1}{2}$$

$$A_s = \frac{504{,}000}{20{,}000 \times 0.92 \times 15\frac{1}{2}} = 1.77 \text{ sq in.}$$

Cost:

Forms—3.08 sq ft @ 70 cents	= 2.16
Concrete—0.81 cu ft @ 50 cents	= 0.41
Steel—$[1.77 + (0.4 \times 1.02)]\ 1.1 \times 3.4$ @ 11 cents	= 0.90
$\frac{2 \times 17 \times 3.6 \times 0.368}{20}$ @ 12½ cents	= 0.28
	$3.75 per lin ft

Floor beam—Ultimate load method

Example 7 is a comparative design by ultimate load theory, using the previously-established load factors, taking bending moments from elastic analysis, and illustrating the design of doubly reinforced beams and tee beams. The algebraic formulas are a trifle awkward for design use but would soon be simplified or converted into charts for office practice. Since no method has yet been formulated for diagonal tension, this design uses $v_c =$

203, and allows 40,000 psi on stirrups at ultimate load. Web reinforcement is only about 10 percent of the longitudinal bars so that any future variations that may be recommended will not greatly affect the general conclusions. This beam is also costed, and, at these units, is just a trifle less expensive than the conventional design, not considering the savings in other portions of the structure previously referred to. It should be noted that the reinforcing steel costs more, the concrete almost the same, and the important factor is the unit price of beam forms.

Example 7—Beam, $L = 19$ ft 0 in.

Live load—$2.4 \times 10 \times 130 = 3120$
Dead load—$1.2\ (10 \times 34.4 + 90) = 521$
3641 psf

$$V = \frac{wL}{2} = \frac{3641 \times 19}{2} = 34{,}590 \text{ lb}$$

$$+M = \frac{wL^2}{16} = \frac{3641 \times (19)^2 \times 12}{16} = 986{,}000 \text{ in.-lb}$$

$$-M = \frac{wL^2}{11} = \frac{3641 \times (19)^2 \times 12}{11} = 1{,}434{,}000 \text{ in.-lb}$$

At end, beam is double reinforced.

$$\frac{-M}{bd^2} = \frac{f_c'}{3} + f_y\, p' \left(1 - \frac{d'}{d}\right) \qquad \text{Try } 8 \text{ x } 13\tfrac{1}{2} \text{ in., } d = 11 \text{ in., } d' = 2\tfrac{1}{2} \text{ in.}$$

Estimating p at about 0.02, $\dfrac{t}{d} = 0.25$, $\dfrac{d'}{d} = 0.227$

$$bd^2 = \frac{1{,}434{,}000}{1000 + 40{,}000 \times 0.02\ (1 - 0.227)} = 893, \quad 8 \times 11^2 = 968 \text{ sq in.}$$

At mid-span, beam is Tee shaped.

$$b \lesssim \frac{19 \times 12}{4} = 57 \lesssim 8 + 8 \times 2\tfrac{3}{4} = 30 \text{ (using } 4t, \text{ not } 8t, \text{ each side)} \lesssim 120.$$

Using 8 x 11 in., $b = 30$

$$z = \frac{d\left[\ (b - b')\ (t/d)^2 + 0.288\ b'\right]}{2\left[\ (b - b')\ (t/d) + 0.537\ b'\right]} = \frac{11\left[22 \times (0.25)^2 + 0.288 \times 8\right]}{2\left[22 \times 0.25 + 0.537 \times 8\right]} = 2.07 \text{ in.}$$

$$M = 0.85\, f_c'\ (d - z)\ A_c$$

$$M = 0.85 \times 3000\ (11 - 2.07)\ (22 \times 2\tfrac{3}{4} + 8 \times 0.537 \times 11) = 2{,}454{,}000$$

$2{,}454{,}000 > 986{,}000$ so f_c is quite low

$$A_s = \frac{M}{f_y\ (d - z)} = \frac{986{,}000}{40{,}000 \times 8.93} = 2.76 \text{ sq in.}$$

Returning to negative moment, for $-A_s'$

$$\frac{M}{bd^2} = \frac{f_c'}{3} + f_y\, p' \left(1 - \frac{d'}{d}\right) = \frac{1{,}434{,}000}{8 \times (11)^2} = \frac{3000}{3} + 40{,}000\, p'\ (1 - 0.227)$$

$p' = 0.0155 \qquad -A_s' = 1.37$ sq in.

For A_s

$$M = A_s'\, f_y\ (d - d') + (A_s - A_s')\, f_y \left[d - \frac{A_s - A_s'}{2\,b} \times \frac{f_y}{0.85\, f_c'}\right]$$

$$1{,}434{,}000 = 1.37 \times 40{,}000 \times 8.5 + (A_s - 1.37)\,40{,}000 \times \left[11 - \frac{A_s - 1.37}{2 \times 8} \times \frac{40{,}000}{2550}\right]$$

$$A_s = 4.37 \text{ sq in.}$$

$$v = \frac{V}{bjd} = \frac{34{,}800}{8 \times \frac{7}{8} \times 11} = 452 \text{ psi}$$

$$v_{\text{¢}} = \frac{3120 \times 19}{8 \times 8 \times \frac{7}{8} \times 11} = 96 \text{ psi}$$

$$\text{If } v_s = 2\tfrac{1}{4} \times 90 = 203,\ a = \frac{19 \times 12 \times (452 - 203)}{2\,(452 - 96)} = 80 \text{ psi}$$

$$A_v = \frac{80 \times 8\,(452 - 203)}{2 \times 40{,}000} = 1.99 \text{ sq in.}$$

Use 19 – # 3 stirrups @ 2 in. and 18 @ 4¼ in. (ACI 806(a))

Cost:

Forms—2.37 sq ft @ 70 cents	= 1.66
Concrete—0.6 cu ft @ 50 cents	= 0.30
Steel—$(2.76 + 0.4 \times 1.62)\ 1.1 \times 3.4$ @ 11 cents	= 1.40
$\dfrac{2 \times 37 \times 2.83 \times 0.368}{20} \times 12\tfrac{1}{2}$ cents	= 0.48
	$3.84 per lin ft

Care should be taken in generalizing from one or two examples. The computer's work seems to be somewhat increased, and more complicated relationships develop, at least until methods are better perfected. With the overload factors used, the beam depth can be considerably decreased if no limitations are placed on depth-to-span ratios. More reinforcing steel is used to accomplish the result, but form costs are a deciding factor in the economics. The possibility of using plastic hinges at the supports has only been suggested and not yet developed.

Exterior column

To illustrate the problem of bending and direct stress, an exterior column of the typical building panel is investigated in Examples 8 and 9. For comparative purposes, it is assumed that the load from the roof (84 psf total load) is approximately one-half that of a typical floor (174 psf total load), and a column supporting the roof and the top floor is taken for study. An allowance is made to represent the wall and spandrel beam, and an overload factor of 1.5 is arbitrarily assumed under the ultimate load method, most of this load being dead load. The moment to be taken into the column was made equal to the positive moment in the beam ($wL^2/16$), a very arbitrary procedure, here assumed for convenience in showing computations and with no suggestion that a frame analysis would produce the same result. The value is doubled because of two beams in a panel, and halved for columns below and above the floor, another purely arbitrary conclusion, but equally fair to the two schemes being compared.

The conventional method is a combination of elastic and plastic design and is not only involved, but open to question; in fact, this is one main reason for going into ultimate load design. The column by ultimate load design is smaller than the conventional one, partly because of less load with

a lighter structure above, partly because of the greater volume in the concrete stress prism, and somewhat because of the relation of overload factors to present safety factors. The costs show the ultimate load design to be $1.63 per vertical foot less expensive than the conventional design. Since the ultimate load design has less formwork, less steel, and less concrete, the conclusions are immediate. In addition, there is an increase in the usable floor space in the building.

Example 8—Exterior column, conventional method

$$1\tfrac{1}{2} \times 2 \times 17{,}680 = 53{,}000$$
$$2 \times 20 \times 900 = 36{,}000$$
$$89{,}000 \text{ lb}$$

$M = (2/2) \times 504{,}000 = 504{,}000$
$e = 5.65$ in. Try 15 x 15 in. with 6 – # 8 bars
$P = 15 \times 15 @ 540 = 121.5$ $\qquad A_c = 225$
$6 \times 0.79 \times 12{,}800 = 60.6$ $\qquad (n\text{-}1)\ A_s = 42.7$
182.1 $\qquad 267.7$

$g = 0.67$ $\qquad p_g = 0.0211$

$$D = \frac{1 + (9 \times 0.021)}{\tfrac{1}{6} + (0.5 \times 9 \times 0.021 \times 0.67^2)} = 5.65$$

$$C = \frac{182.1}{267.7 \times 1350} = 0.505$$

$$N = \frac{182.1}{1 + \dfrac{(0.505 \times 5.65 \times 5.65)}{15}} = 88.5 \text{ kips}$$

Cost:
Forms—5 sq ft @ 60 cents	= 3.00
Concrete—1.56 cu ft @ 50 cents	= 0.78
Steel—6 × 2.67 × 1.20 @ 11 cents	= 2.12
	$5.90 per lin ft

Example 9—Exterior column, ultimate load method

$1\tfrac{1}{2} \times 2 \times 34{,}590 = 103{,}800$ $\qquad M = 986{,}000$
$1.5 \times 2 \times 20 \times 900 = 54{,}000$ $\qquad e_t = 10.25$; $(e = 6.25)$
$157{,}800$

Try 13 x 13 in. with 4 – # 7 bars

$$N = \frac{2\,(d - d')\,A_s f_y}{e_t - d + \sqrt{(e_t - d)^2 + \dfrac{2\,(d - d')\,A_s f_y}{0.85\,f_c' b}}}$$

$$N = \frac{2 \times 8 \times 1.2 \times 40{,}000}{10.25 - 10.5 + \sqrt{(-0.25)^2 + \dfrac{2 \times 8 \times 1.2 \times 40{,}000}{0.85 \times 3000 \times 13}}} = 168{,}000 \text{ lb}$$

A 13 x 13-in. column with 4 – # 6 bars carries only 145,000 lb.

Cost:
Forms—4⅓ sq ft @ 60 cents	= 2.60
Concrete—1.17 cu ft @ 50 cents	= 0.59
Steel—4 × 2.044 × 1.20 @ 11 cents	= 1.08
	$4.27 per lin ft

SUMMARY

The foregoing resumé of the high spots of some comparative designs indicates that there is not too much difference in computing time. A typical structure works out with somewhat smaller (and more flexible) members. Comparative costs for the members themselves show savings by ultimate load design, with any saving in architectural construction also in favor of ultimate load design. Other points to observe are that better procedures for handling diagonal tension and bond will have to be worked out, possibly with some cognizance of the longitudinal bars as suggested by Moretto[2] and Clark,[3] and, undoubtedly, with relatively low values for v_c, as suggested by Hognestad;[4] again limitations will have to be developed for control of deflections in flexural members, probably with a requirement that deflections be computed and kept within prescribed limits based upon acceptable sags; overload factors will have to be evaluated quite carefully for different usages; controlled concrete will very likely be the only acceptable method for proportioning, with the required cylinder strength 15 percent above the design value. The ultimate load method is particularly important in reviewing special problems where the designer wants the most realistic appraisal of actual conditions that he can obtain.

CONCLUSIONS

Designers will want to know why they should change techniques and what the results might be, so a few questions are raised and answered briefly.

1. Will ultimate design work? Of course it will, the values are obtained from test data. It was the difficulty of trying to reconcile tests to failure with results obtained by projecting the uniformly-varying stress theory that brought ultimate load design into being.

2. Will ultimate load design decrease the cost of structures? Probably, to some extent. Engineers used to estimating realize that unit costs are in a state of constant flux, and no exact conclusions can be drawn. Contractors in the same city will vary considerably in unit costs of operations, though they balance out fairly well in the grand total. It appears likely that some savings can be made by the careful designer, partly in the structural framework itself and partly in the architectural coverings.

3. Will the amount of reinforcing steel required per structure change much? Possibly a slight increase, but not too much. While ultimate load design carried to the capacity of the compressive concrete or even augmented with compressive bars can produce very slender members, steel is relatively expensive and designers will soon learn an economic balance between the component parts and will use slightly deeper sections than the absolute minimum, so the quantity of reinforcing steel will probably be somewhat increased but only by a small percentage.

4. How will the amount of cement per structure be affected? Probably the total will be about the same or very slightly decreased. The yardage of concrete will decrease with smaller members, but the requirements of carefully controlled concrete will undoubtedly increase the cement content per yard of concrete in most cases.

5. Is there much more labor for the computers? No, not enough to think about. The work will be decreased as methods are improved. Designers will have a much more realistic background against which to evaluate their judgment, and computing time will be offset by the time saved in thinking about load conditions, providing a balanced structure and keeping within prescribed clearance lines.

6. Will building codes permit such analyses? Undoubtedly, as skill and experience develop. Once the engineering fraternity gives its unqualified approval to the procedure; methods, codes, textbooks, tables, and diagrams will follow along just as rapidly as the previous transition from the early parabolic stress prism to the conventional triangular one.

7. Can the ultimate load method be taught in university courses? From the author's experience, it is believed that you can teach clearly any theory in which you have confidence, have proved by tests, and thoroughly understand.

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Discussion

By PHIL M. FERGUSON*

Mr. Reese has shown that it is practical to design by ultimate strength methods. The writer has been asked to discuss such design from the viewpoint of the educator and his classroom instruction of student engineers. As Mr. Reese states, there is no question about the teachability of ultimate strength design, if the teacher really understands and *believes* in it. The pertinent questions are two. First, how can ultimate strength theory be presented effectively. Second, do new factors demand such time consuming treatment as to upset the usual time allotment for this subject. Experience at the University of Texas during the past two years provides partial answers.

Prior to 1950 ultimate strength design had been left to the graduate course level, except for a single parabolic stress distribution problem used to point up a discussion of safety factors.

In 1950 it was decided that reinforced concrete might be better understood by senior students if the first introduction of the subject frankly faced the fact that actual bending stresses under ordinary working loads could *not* be calculated. The beginning course was changed to start with consideration of ultimate strength, this approach being introduced in class notes as follows:

> Although present day specifications emphasize stresses at working loads and present day calculations pretend to calculate these stresses, laboratory tests of reinforced concrete beams show that actual deformations and stresses at working loads only faintly resemble our calculated values. In other words, present day specifications and the corresponding standard calculation procedures are not representative of the real action of reinforced concrete beams under working loads. Neither are they representative of ultimate load conditions. They are totally artificial and have been applied with allowable unit stresses empirically adjusted to give safe results.

*Member American Concrete Institute, Professor and Chairman, Civil Engineering Dept., University of Texas, Austin.

Probably it is not feasible (and it may not even be possible) to do more than crudely estimate the approximate magnitude of real stresses under working loads. Certainly these depend upon shrinkage and upon how much cracking has been induced by previous loadings.

Fortunately we are able to predict or calculate the ultimate strength of reinforced concrete beams with much better accuracy, although questions still remain even in this area.

Although only four or five classes are now devoted to ultimate strength discussion, this realistic approach has proved quite successful. Topics are discussed in this order.*

1. Strength of concrete.
2. Significance of low tensile strength.
3. The resisting couple in a beam. Without any discussion of the distribution of compressive stresses, the relation $M = C \times jd = T \times jd$ is pictured, j and d being defined in standard terms.
4. The complete stress-strain curve for concrete in compression. Attention is called to the portion beyond the peak stress f_c'.
5. Probable distribution of compressive stress in a beam. Sections of the stress-strain curve are used to indicate the shape of the stress block acting at small loads (straight line), moderate or working loads (slightly curved), near ultimate load (parabolic), and at ultimate load (Whitney's presentation with extreme fiber stress less than maximum stress). All of this is specified to be at first loading without prior shrinkage. (At this point the paragraphs quoted above are introduced.)
6. Failure of a reinforced concrete beam in bending.

a. Tension failure—The rising neutral axis as steel reaches yield point slightly increases the jd lever arm. Secondary failure occurs in compression. Deflection is greatly increased near failure. No mathematical analysis is presented.

b. Beams equally likely to fail in tension or compression.

c. Compression failure—The yielding of the concrete lowers the neutral axis and significantly increases the area under compression. Failure is sudden without much warning.

d. The small effect of shrinkage and creep on ultimate strength.

At this stage the student has a very good picture in mind. Definite relations for practical use are then introduced.

7. Whitney's ultimate strength relations.

a. Beams failing in compression—$M_{ult.} = (1/3)\, f_c' \times b \times d^2$, given as an empirical relationship. (Also see c below.)

b. Under-reinforced beams—Whitney's rectangular equivalent stress block deep enough to provide $C = A_s f_y$, and the corresponding moment.

c. Over-reinforced beams—No advantage accrues from excess steel. The maximum depth of stress block is empirically determined as $0.537\, d$, which is used for balanced beams.

8. Parabolic stress distribution—This is given for an over-reinforced beam as an example of an older approach to ultimate strength.

With this approach, students seem to understand what is happening. There should be no complications in following with double-reinforced beams, T-beams, columns, and combined direct stress and bending under an ultimate strength theory. Teachability can doubtless be improved by some slight alterations of form. For example, double-reinforced beam formulas will look simpler when the resisting moment is broken up into two couples, as is currently done with straight line methods.

*Copies of these class notes are available upon request to the writer.

The University of Texas undergraduate student does not follow ultimate strength theory beyond the simple rectangular beam. Instead he is told that standard specifications have not accepted ultimate strength design (except for columns and beam compressive steel) and hence the remainder of his course will deal with the simple straight line methods of the ACI Building Code. The impatience of youth is usually apparent at this point and the teacher is considerably on the defensive in explaining the gradual evolutionary process of revising specifications in a democratic society.

In contrast, a teacher actually has difficulty in fully defending our present dual standard specification. With working load analysis for beams and ultimate strength design for columns, combined direct stress and bending is neither one nor the other, but a patchwork in between. One must tell the student he *cannot* calculate the steel or concrete stress in a centrally loaded column because of serious shrinkage and creep complications. Two classes later, for the more complicated case of direct stress and bending, the ACI Building Code *requires* him to calculate an extreme fiber stress. Maybe our Building Code Committee should be asked to add a preface that will explain this as a purely dogmatic or artificial approach to a complex situation not yet clarified by test data. Although the writer realizes that specifications evolve slowly, he cannot pass this opportunity without saying that he greatly dislikes having to teach students to calculate extreme fiber stresses on eccentrically loaded columns. The calculations involve too much algebra and too little truth.

Unfortunately, ultimate strength design procedures also have their patchwork spots. Whitney's relation for an eccentrically loaded column with small eccentricity is a sample of a purely arbitrary correlation. The teacher will have to develop a more logical derivation or else classify his material into two groups, one group based on consistent logic, and a second group simply convenient interpolations.

There is no doubt that ultimate strength design can be taught as easily as our present methods. There will be no student resistance to ultimate strength design unless the teacher implants such resistance. Of course, to teach such methods well, a teacher must know more about how concrete really acts.

The writer believes that the time required to teach one method will be about the same as for the other. He worries a little about the transition period when he might be expected to teach both methods in the same limited time. He also notes from Mr. Reese's paper that more attention will have to be directed towards slab and beam deflections, subjects which receive scant attention now. More fundamental study and research will certainly be needed in this area.

Two other theoretical matters will worry teachers considerably. One is the dependable diagonal tension strength when bending deformations are large. The other is whether design bending moments should be based on limit design methods or upon elastic analysis. Further studies are needed.

Discussion of a paper by R. C. Reese:

Practical Design at Ultimate Loads*

By H. J. COWAN and AUTHOR

By H. J. COWAN†

The writer was interested to find that Mr. Reese's examples show relatively small or even negative savings resulting from ultimate strength design. This appears to be partly due to choice of load factor and partly to the selection of the examples. The writer has compared elsewhere the working strength and ultimate strength methods in a number of examples[1-3] in which much larger economies were obtained.

The load factor was chosen so as to give the same safety margin as the working stress method. The British code of practice specifies working stresses for the steel equal to one-half the yield stress, and working stresses for the concrete equal to one-third the 6-in. cube crushing strength (corresponding roughly to 1/2.5 of the crushing strength of the American standard cylinder). The load factor was therefore taken as 2, which was considered adequate to cover uncertainties in loading conditions, approximations made in design theory, and imperfections in workmanship, and variations in the quality of the *steel*. This was supplemented by an additional factor of safety of 1.25 to cover the greater variation in workmanship and quality of the *concrete*. This additional factor of safety was introduced by specifying a crushing strength for the control cylinders 25 percent higher than that used in design calculations; the concrete mix would be designed to give this higher strength.

Using this method, the ultimate strength design is always more economical than the working stress design, although the saving is negligibly small in T-beams and under-reinforced rectangular beams with tension reinforcement only. On the other hand, substantial reductions in the weight of steel are obtained for all beams containing compression reinforcement. The same applies to rectangular beams with tension reinforcement only, which have a percentage of steel above p_w and below p_u, where p_w and p_u are the percentages of steel for balanced design obtained from the working stress and ultimate strength theories respectively; for a 3000-psi concrete and intermediate grade steel p_w would be approximately 1¼ percent and p_u approximately 3¾ percent. In some examples a saving in steel of almost 30 percent was recorded.

*ACI JOURNAL, June 1952, *Proc.* V. 48, p. 865. Disc. 48-55 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Lecturer in Civil Engineering, University of Sheffield, Sheffield, England.

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†Lecturer in Civil Engineering, University of Sheffield, Sheffield, England.

Although the exact figures would be modified if load factors appropriate to American conditions and materials are used, the saving in steel would be of the same order for these two types of section.

It is worth noting that the balanced percentage is always much higher in ultimate strength design than in conventional working stress design, this higher percentage being in accord with the known experimental data.[1] However, whereas the balanced percentage obtained from the working stress method is often also an economical percentage, the balanced percentage corresponding to ultimate strength is generally above the economic limit for slabs and short span beams. There is, however, no special virtue in the use of a balanced design, and slabs designed on an ultimate strength basis should always, in the writer's opinion, be under-reinforced. If the thickness of slabs is kept at the same figure as for working stress design, there is a small saving in steel due to the greater length of the moment arm in the ultimate strength equations.

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AUTHOR'S CLOSURE

Dr. Cowan's discussion is a helpful contribution. As mentioned in the original paper, one of the items which must be settled before starting the practical design is the load factor. Prof. T. Y. Lin in "Load Factors in Ultimate Design of Reinforced Concrete"* points out many of the items which must be taken into account in selecting a load factor. Dr. Cowan mentions the use of a factor 2, which, the author assumes, applies to live load only, probably with a lesser factor for the dead load. However, he recommends requiring a concrete whose crushing strength is 25 percent higher than its design strength, having in mind that concrete is a more variable material than steel. It was interesting, therefore, to note that his practice is not too greatly divergent from that used in the original paper.

Dr. Cowan advises that he has obtained greater savings between ultimate strength designs and working stress designs than was the case in the examples selected. The examples were intended primarily to show computers actual office procedures of designing at ultimate loads and to suggest that there was no more effort involved and a more realistic appraisal of the finished result. The cost comparisons were presented mainly to show how the designer should go about making his own analyses. Costs are an extremely tricky and variable factor. They not only vary with labor rates, but also with the productivity of labor. They vary not only between one location and

*ACI JOURNAL, June 1952, *Proc.* V. 48, pp. 881-900.

another, but between individual contractors working in the same location. An experienced engineer will not attempt to gain more than a rough impression from someone else's generalized cost comparisons.

However, the author agrees with Dr. Cowan, that the illustrations chosen in the original paper were of such a conventional short-span structure and the load factors were chosen so conservatively that the comparisons do not reflect the full savings that can often be attained by a carefully prepared ultimate load design.

"One reason that American practice will not show abnormally large savings in the case of beams with compressive reinforcement is the stipulation in "Building Code Requirements for Reinforced Concrete Construction, ACI 318-51," that only half as much compressive reinforcement need be furnished as required by elastic analysis under the conventional method to make some allowance for plastic time-yielding of the concrete."

Like all other cost analyses, this paper can only be a guide to methods and procedures, and engineers who are not familiar with ultimate load design are urged to try a structure or two and make their own analyses and comparisons. Finally, ultimate load design will be found particularly useful in rating the safe carrying capacity of existing structures or in trying to report upon the causes and reasons for failures.

Title No. 48-56

Load Factors in Ultimate Design of Reinforced Concrete*

By T. Y. LIN†

SYNOPSIS

Reasons for margins of safety are discussed. Shortcomings of the present method of allowable stresses are enumerated and bases for the choice of load factors in ultimate design are explained. Tentative load factors are proposed.

INTRODUCTION

According to our conventional method of reinforced concrete design, a normally maximum service load is specified as the design load, the elastic theory is followed in the computation of internal stresses, and a table of allowable values is set up as the maximum limits. In following such a procedure, a margin of safety is provided solely in the allowable stresses, while no allowance is made directly for the possibility of abnormally heavy loads.

In ultimate design, we deal with ultimate strengths instead of allowable stresses. To provide a margin of safety, especially to take care of extra heavy loads, it is necessary that the specified service loads be multiplied by load factors and the products be used in design. The considerations involved in the choice of these load factors together with some proposed values for them are discussed here.

When the elastic theory was first introduced into our design specifications, there were little data concerning the actual reserve strengths in structures so designed. There were still less data for forecasting possible future loads which might act on the structures. As more and more structures were designed and built, and as materials and methods of construction were constantly improved, allowable stresses were gradually raised to the present values although there were no basic changes in the procedure of design. Further experience with these structures showed that many of them did often carry loads over and above the design loads without causing any noticeable damage, although in rare instances, some parts of certain structures did occasionally show signs of distress and once in a long while, an overloaded structure would actually fail and completely collapse. Such usually large though often nonuniform overload capacity of structures has led engineers

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†Member American Concrete Institute, Associate Professor of Civil Engineering, Associate Engineer, Institute of Transportation and Traffic Engineering, University of California, Berkeley.

to believe that for safety as well as economy, a more logical provision for overloads should be devised.

The primary purpose of ultimate design is to obtain a more uniform overload factor when the structure is loaded to near its ultimate strength. This does not imply that overloads should be more encouraged on structures so designed. On the contrary, one major reason for the use of load factors in ultimate design is to avoid the raising of nominal design loads to which service loads should be generally limited. In case of emergency, however, occasional overloads may be permitted with greater assurance on structures designed by the ultimate theories because the weakest links which would have resulted from conventional method of design would now have been strengthened by ultimate design.

Due to the nonproportionality of member strength to its stress, designing by ultimate strength theories will naturally not result in uniform stresses. Similarly, designing by allowable stress method will not result in uniform strengths. Hence it becomes a problem of logic and judgment whether the primary concern in design should be to secure uniform strengths in a structure under overload or to obtain uniform stresses in it under service loads.

WHY AND HOW TO PROVIDE MARGINS OF SAFETY

To arrive at a correct determination of margins of safety, it is necessary to first probe into the reasons for the use of such margins. It is known that margins of safety are primarily required to provide for the uncertainties of loads, especially to take care of overloads. However, they are also required to allow for other unknown items not involving loads. Furthermore, because of other practical concepts affecting judgment in design, often-times it is also deemed desirable to vary the margins of safety based on a sense of relative values and significance. Thus, the reasons for margins of safety can be outlined under three headings.

Uncertainties of loads

1. Accuracy of design loads in representing present actual loads, *e.g.*, an H20-S16 design truck for highway bridges can be greatly exceeded by an actual heavy truck.
2. Assumptions regarding load distribution, *e.g.*, the actual load distribution between girders supporting a slab may be different from the assumed distribution.
3. Possible future increase in magnitude of loading and change in type of loading, *e.g.*, overloading of warehouses by heavy storage or concentrated and moving loads such as fork trucks.
4. Frequencies of loading—increase in frequencies of loading might result in fatigue failures.

Items not involving loads

1. Accuracy of prevailing methods of analysis, *e.g.*, in a building of equal bays, the live load moment in columns due to unbalanced loading is usually neglected in design.
2. Reliability of the strength of materials, *e.g.*, it is generally believed that the strength of concrete may not be as consistent as that of steel, although by application of modern methods of control to mix design, tests indicate no greater variation in strength of concrete than that of steel.
3. Possibility of deterioration, *e.g.*, exposure to sea water.

Practical concepts affecting desired margin of safety

1. Importance and service nature of structure, *e.g.*, a monumental structure may require a greater margin of safety.
2. Seriousness of failure, *e.g.*, a main column continuous from story to story may require a greater margin of safety than a post in a single story only.
3. Suddenness of failure, *e.g.*, a spiral column is tougher than a tied column; it gives warning before complete collapse and hence may require a smaller margin of safety.
4. Economics of provision for additional safety, *e.g.*, thickening a slab often provides more rigidity and safety in concrete at very little expense, because of resultant saving in steel.

Having outlined the different reasons for margins of safety, we are now in a position to discuss methods for providing them. There are several different ways to provide margins of safety. They can be provided solely in the stresses, by using suitable values of allowable stresses. They can be provided solely in the loads, by using suitable load factors. They can be provided solely in the dimensioning of structural parts, by adopting sizes greater than computed. They can also be provided in any combination of the above methods: some in the stresses, some in the loads, and some in the design dimensions.

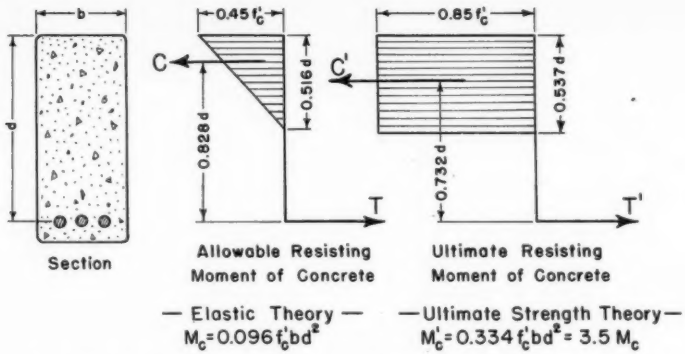
The proper criteria for a satisfactory method of providing safety margins are these:

1. Correctness, *i.e.*, suitable margins for all parts of all structures under critical conditions.
2. Simplicity, *i.e.*, easy and convenient application of the method in design.
3. Adaptability, *i.e.*, flexibility in the extension and application of the method to special cases or unconventional structures. This requires logical reasoning in the process of safety provisions so that the designer will visualize as much as he can where and why the margins of safety are being provided. This will enable the designer to properly make adjustments to suit his particular problem.

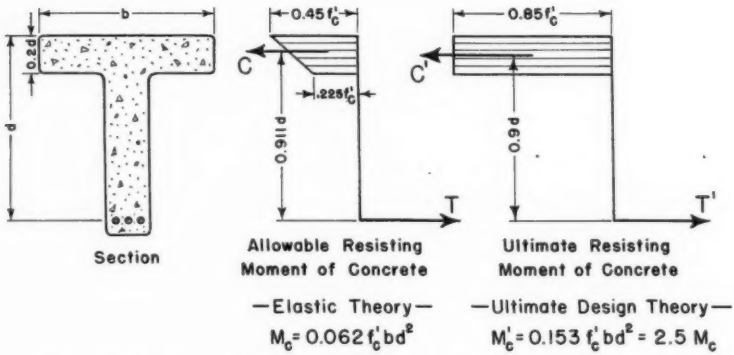
With these criteria in mind, it must first be stated that as far as correctness is concerned, any of the above methods can be made to work correctly, provided proper margins are chosen. In other words, correctness could always be attained if a great number of margins could be specified or chosen to suit all different cases. However, such extensive and all-inclusive specifications being quite impracticable, the method of providing safety margins should be as logical as we can make it so that specifications will yield reasonably correct results for all cases without involving unnecessary complications. By providing a margin of safety wherever it belongs and wherever it is needed, the method will be more realistic and hence more adaptable. Although it was the general impression that such a procedure might entail more computation work than the usual practice of providing a gross margin in only one item, a detailed comparison will reveal that such a logical and realistic approach is in fact the most practical and simplest method, especially in connection with ultimate design of reinforced concrete.

LOAD FACTORS *vs* ALLOWABLE STRESSES

Our conventional method puts a margin of safety solely in the allowable stresses. The shortcomings of such a procedure lie in its inability to provide



(a). Rectangular Beam—Overload Factor = 3.5 in concrete



(b). T-Beam — Overload Factor = 2.5 in concrete (neglecting web)

Fig. 1—Different load factors for rectangular beam and T-beam

a uniform overload capacity for all parts and types of structures. The first reason for such a variation is the nonproportionality of stress to strain in the plastic range as advanced in ultimate strength theories. As an example, consider a rectangular beam and a T-beam, both being designed for an allowable stress of $0.45f_c'$ in the concrete (Fig. 1). Ultimate strength theory shows that the concrete resisting capacity will increase about 3.5 times for the rectangular beam while only about 2.5 times for the T-beam.

Such inequalities of safety are sometimes recognized in conventional design and equalized by allowing different stresses. For example, consider a column under axial load. Its ultimate stress is generally about $0.85f_c'$. If

the allowable concrete stress in the column was also $0.45f_c'$, as in a beam, the overload factor would be only $0.85f_c'/0.45f_c' = 1.9$, as contrasted to 3.5 previously mentioned for a rectangular beam. To obtain a degree of safety comparable to that in a rectangular beam, it is necessary in conventional design to allow a lower stress for axially loaded columns such as $0.225f_c'$ for spiral columns or $0.18f_c'$ for tied columns. On the other hand, if a column is subjected to an eccentric load, partial beam action will exist and a higher unit stress can be permitted. Such a method of variable allowable stress is one way to circumvent the shortcomings of the allowable stress method. Although it can be made to work it is admittedly rather inconvenient.

As another example of the nonproportionality of overload factors to stresses, take a prestressed concrete beam. When load on the beam increases from service load to the ultimate load, the lever arm of tension in the wires increases appreciably. Hence, although most designs allow a margin of safety of only about 0.6 in the allowable stress for wires, the ultimate moment capacity will provide an actual overload factor of 2.2 (Fig. 2b). Compare this to a heavily reinforced concrete beam (Fig. 2a) which may have a margin of safety of 1.0 in the allowable stress, but may only provide an overload factor of 1.8. This makes it difficult to obtain equal safety by the method of allowable stresses; but such equality can easily be realized by using proper load factors in ultimate design.

The second and perhaps most prominent objection to providing safety solely in the allowable stresses is pointed out by advocates of balanced design. The allowable stress method provides the same margin of safety for both dead and live loads. Since the dead load of a structure increases but very little, the reserve strength for dead load can usually be utilized for further live load increases. Thus the capacity of a structure to carry excess live load is dependent on the ratio of dead load to live load, resulting in a variable capacity for carrying excess live loads. This has sometimes ended in the premature failure in portions of structures with low DL/LL ratio, such as floor slabs of bridges and buildings. At the same time, other parts of such structures are often unnecessarily strong. The most direct remedy for this is to place the desired margin of safety in the loads where it belongs.

To do justice to the conventional method of design, it must be pointed out that it is also possible, although not entirely convenient or logical, to achieve balanced design without using load factors. In fact, some of our present specifications have included such provisions in the following ways.

1. Double live load clause in bridge specifications. This provides for the checking of structures with twice the design live load and not to exceed 50 percent overstress. It is equivalent to using balanced design for certain parts of the structures.
2. Dual stress method of design. In the design of some structures, two sets of allowable stresses are sometimes specified: one lower set for live load and another higher set for dead load. This is equivalent to the use of a higher load factor for live load than for dead load.
3. Discounting dead load stresses. The computed stresses due to dead load are discounted by a certain percent before being added to the live load and used in design.

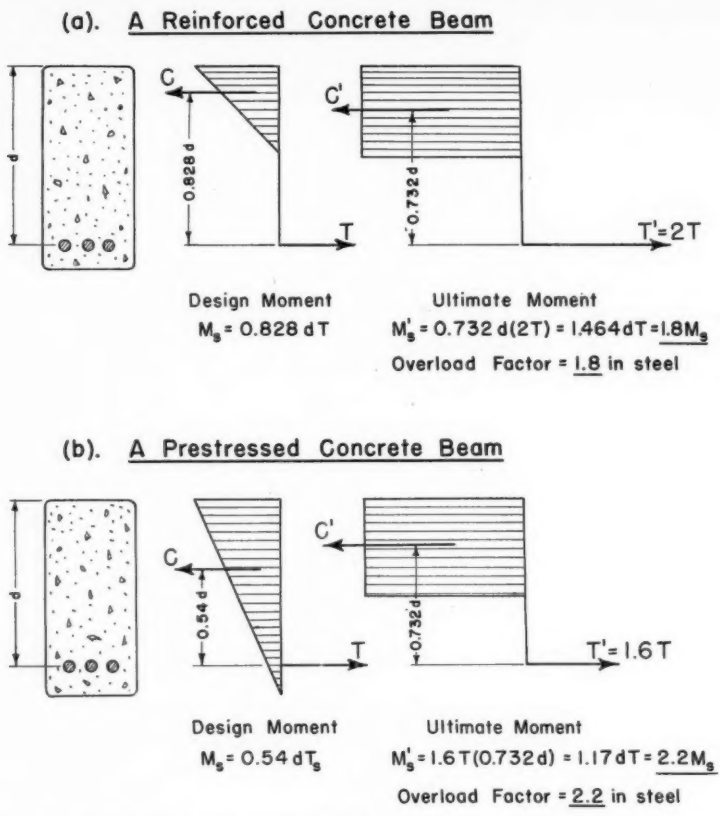


Fig. 2—Different load factors for a reinforced and a prestressed concrete beam

For a proper discount and a corresponding set of allowable stresses this could yield exactly the same results as the dual stress method. Such a procedure may not sound realistic, but it avoids the use of two sets of allowable stresses and is simpler in that sense.

4. Rating old structures. Higher stresses are allowed for old structures which are already carrying overloads and are not expected to carry still heavier loads. Since in such a rating there usually has been no increase in dead load but only increase in live load above the design values, this method amounts to the application of different load factors for dead and live loads.

It can be seen that while some of these modifications appear unrealistic and others are relatively inconvenient, their adoption was justified by the particular requirements of the situation, generally in an effort to obtain a more reasonable design or a more logical review. Resort to such methods

clearly indicates the inadequacy of the conventional method of design and suggests a reappraisal of the situation.

VARIABILITY OF LOAD FACTORS

To choose load factors correctly, the complexity of the problem must be first comprehended. First of all, load factors will usually be different for different kinds of loads: dead load, live load, impact loads, snow loads, ice loads; strain loads such as temperature and shrinkage; lateral loads such as wind or earthquake. Second, load factors may vary with different combinations of loads: *e.g.*, load factors for live load might be smaller when used in combination with wind loads, load factor for dead load may be different when combined with live load of the opposite sign. Third, load factors may vary with the nature of the load: whether dynamic or fatigue producing loads. Fourth, load factors may vary with the type of structure: whether a bridge, a building, or perhaps a tower or a tank.

The above variables are primarily a function of the load itself and are not related to stresses or strengths. Hence they can best be taken care of in the loads by specifying different load factors. There are, however, other factors previously brought out which have no relation with loads, and can be much more directly expressed in terms of strengths. This makes it desirable to provide some margin of safety in the ultimate strength formulas rather than exclusively in the load factors. To many advocates of ultimate design, to be true to the principle of *ultimate* strengths, it would appear at first sight that load factors should be the only place for a margin of safety and only the actual ultimate strengths should be used in design. However, after a careful study of the problem, the writer has come to believe that tested ultimate strengths cannot be consistently used as the values in design.

By segregating the margin of safety for strengths from that for loads, it becomes possible to specify load factors without a complete list of ultimate strength formulas. Any desired adjustment in the strength formulas of various members and materials can be made independently later on as the tested formulas become available. In fact, it would even be possible to apply load factors together with the elastic theory if so desired and if a new set of allowable stresses be specified. This means that the use of load factors, one of the essential features of ultimate design, can be thus applied to the design of all structures, although ultimate strength formulas may be available only for concrete. This is an important consideration when taking into account the combination of several materials in one structure. In fact, although we do not have sufficient data regarding the practice in other countries now using ultimate design, it is reasonable to conceive that they are probably doing the same thing, *i.e.*, adopting balanced design for all structures by the use of load factors without incorporating a complete list of ultimate strength formulas. So far as reinforced concrete is concerned, it is the writer's opinion that many ultimate strength formulas have already been formulated in this

country and it is not premature to start both changes—the use of load factors and ultimate strength formulas—at the same time.

It must also be pointed out that ultimate strength formulas are derived from test results. Such results may vary depending upon interpretations and formulations. These formulas generally represent only the average tested strengths, while sometimes the minimum tested strengths would be a better criterion for design. Because of these difficulties it is considered impractical, at least at this moment, simply to adopt the tested ultimate strengths as the allowable strengths in design. Thus, it is the writer's opinion that, in addition to applying load factors to the service loads, tested ultimate strengths must be transformed to allowable ultimate strengths to be used in design. In other words, a margin of safety should not always be provided only in the loads, but also in the strengths whenever necessary.

Additional reasons for providing such a margin of safety in the ultimate strengths are:

1. The margin of safety for concrete usually differs from that for steel, because of the possible variation of concrete strengths and of the lack of toughness in concrete. Thus a difference in the margins of safety is considered desirable, at least to a certain degree. Since we cannot logically use one set of load factors for concrete and another for its reinforcing steel, we will have to provide for their difference in their respective allowable strengths.

2. It is the present practice to provide different margins of safety for different grades of reinforcing bars because of their varying ductility and also because of deflection and crack limitations. Thus at the present time the margins of safety for three grades of reinforcing steel differ as listed in Table 1.

While this divergence may not be entirely justified, it is probably one of those items which cannot be changed without careful consideration. To maintain these differences, it is again necessary to modify the tested ultimate strengths by different amounts.

3. Reinforced concrete is often used together with other materials such as structural steel or timber. At present different margins of safety are being used for these materials as shown in Table 2.

These different margins of safety may again be necessary in view of the different behavior of the various materials at or before failure. In timber, for example, a greater portion of its strength before complete rupture simply cannot be utilized because of excessive deflections under high stress. Only

TABLE 1—MARGINS OF SAFETY FOR DIFFERENT GRADES OF REINFORCING STEEL

Grade of steel	Structural	Intermediate	Hard
Yield point, psi	33,000	40,000	50,000
Allowable strength in bending, psi	18,000	20,000	20,000
Margin of safety	0.8	1.0	1.5

by providing proper margins in the strengths can we equalize the different degrees of safety desired for different materials. If only the ultimate strengths were to be used it would be necessary to make adjustment in the load factors, using one load factor for concrete and another for timber or steel although they are all used in one structure. This would appear illogical and inconvenient.

4. Different reinforced concrete members may need different margins of safety due to approximations currently used in our methods of analysis, *e.g.*, our method is generally more exact for beams than for columns as previously mentioned. Again, different margins of safety are now used for tied and spiral columns. It would be unreasonable to use one set of load factors for tied and another for spiral columns, one set for beams and another for columns.

5. It may be desirable to reserve some strength to take care of wind or other lateral forces. Hence if tested ultimate strengths are used when considering wind, smaller strengths will need to be used when not considering wind, unless two different sets of load factors are to be specified.

It must be remembered that in speaking of ultimate design we are actually trying to design a structure for its critical point of failure when subjected to some excessive loads, whether remote or imaginary. That point of failure is an important consideration. It could be the complete collapse of a building, or it could be merely the cracking of a storage tank. Very often, the mere permanent set or excessive deflection of certain portions of a structure may mark the limit to which the engineer would define as failure. It is because of these different requirements of structures that we cannot simply set the point of complete collapse as the only criterion for design, because long before its collapse the structure would have been rendered useless.

It is sometimes believed that a dual provision of margins of safety might entail too much work in design or might result in excessively conservative structure. This is not true. Concerning simplicity, since the specifications will only carry one set of ultimate strength formulas, whether tested or allowable, there will be no additional complications involved in using the allowable formulas, although as a matter of information it might be desirable

TABLE 2—MARGINS OF SAFETY FOR DIFFERENT MATERIALS OF CONSTRUCTION

	Reinforcing steel	Structural steel	Timber
Ultimate strength, psi	55,000–80,000	60,000–72,000	6000–14,000
Yield point	33,000–50,000	33,000	——
Allowable stress, psi	18,000–20,000	18,000–20,000	1200–2400
Margin of safety	0.8–1.5*	0.6–2.6†	4–6

*Failure of reinforced beams generally starts at the yield point of reinforcing bars resulting in eventual crushing of concrete. Hence yield point of steel is used for computing the margin of safety.

†Structural steel may fail either at the yield point in compression or at the ultimate strength in tension; so both are used in computing the margins of safety.

to list the tested ultimate strength formulas simply for reference. Concerning the possibility of excessive safety, it is entirely dependent on the proper choice of correct values of margins. Dual margins might even result in less conservatism if small values are chosen.

ALLOWABLE ULTIMATE STRENGTHS

Because of the above considerations it is deemed necessary to adjust margins of safety for different materials and members by transforming the tested ultimate strengths to allowable ultimate strengths. While it is not the purpose of this paper to discuss the ultimate strength formulas, it is necessary to choose some ultimate or allowable ultimate strengths in order to decide on load factors. For convenience in discussion, consider intermediate grade steel bars in beams whose failure starts in steel at its yield point. This is chosen because it is the most common case in reinforced concrete construction and will serve well as our basis for discussion.

Beams—Failure starting in steel

(*a*) *Intermediate Grade*—Tested ultimate strength of such beams is often given by the formula $M = K_1bd^2$. Let us set this tested ultimate strength as the allowable ultimate strength for the combination of dead, live, strain, and wind loads, all multiplied by the respective load factors. This would tend to indicate that for this case we are dealing with true ultimate design, equating ultimate loads to the ultimate strength. Then, when not considering wind load, the allowable ultimate strength should be reduced by a factor. To be consistent with the usual provision for a one-third increase in allowable values when considering wind loads, the allowable ultimate strength should be ¾ of the above, *i.e.*, $M = 0.75K_1bd^2$ when not including wind. It is understood, as usual, that structures should be designed to withstand both conditions, whether with or without wind, and the critical condition should govern. This proposed allowable ultimate strength formula would be equivalent to designing for about 30,000 psi without wind and about 40,000 psi with wind.

(*b*) *Structural Grade*—Having decided on the allowable ultimate strength for the basic case, consider structural grade steel. This has a yield point of 33,000 psi as against 40,000 psi for intermediate grade, but under present codes it has an allowable stress of 18,000 psi—a margin of safety of 0.83 as against 1.00 for intermediate grade. This difference in the margins has been justified by the toughness and ductility of the structural grade. To retain this same relation the allowable ultimate strength for structural grade should be greater by the ratio 2.00/1.83 = 1.08. Hence, if the tested strength is given by $M = K_2bd^2$, the allowable ultimate strength should be $1.08 \times 0.75K_2bd^2 = 0.81K_2bd^2$, without wind, or $M = 1.08K_2bd^2$ with wind. Although this results in an allowable ultimate strength 8 percent above the ultimate, it might be permissible because of co-existence of several high load factors.

(*c*) *Hard Grade*—It has been the practice to provide a greater margin of safety for hard grade steel. Since the yield point is 50,000 psi while the

allowable stress is only 20,000 psi, a margin of safety of 1.5 is obtained as compared to 1.0 for intermediate grade; hence the allowable ultimate strength should be $M = 2.0/2.5 \times 0.75K_3bd^2 = 0.60K_3bd^2$ when not considering wind, in order to maintain the same relative margin of safety. If such additional safety is not considered necessary, then the same formula as for intermediate grade can be used, *i.e.*, $M = 0.75\ K_3bd^2$.

It is well-known that ultimate design has already been applied to some extent in the design of concentrically loaded columns. The ultimate strength of columns is given by the formula $N = 0.85f_c'A_g + f_yA_s$. The present allowable strength is given by $N = 0.225f_c'A_g + 0.40\ f_yA_s$ for spiral columns. Allowing a 50 percent increase similar to the case of intermediate grade steel in beams, we have $N = 0.34f_c'A_g + 0.60f_y'A_s$ as the allowable ultimate strength for spiral columns when not considering wind. For tied columns, following the present practice, 80 percent of those of spiral columns should be used, *i.e.*, $N = 0.27f_c'A_g + 0.48f_y'A_s$.

By listing these formulas in Table 3, we can compare the ratio of tested ultimate strength/allowable ultimate strength for the various cases.

It is to be noted that the ratio varies from 1.23 for structural grade steel in beams to 3.15 for concrete in tied columns. While much of this variation can be justified, some of it probably needs revision. Although it is beyond the scope of this paper to discuss such revisions, by the adoption of load factors together with allowable ultimate strengths these discrepancies will be brougnt to light and may in the course of time be gradually corrected as more data and studies become available. These allowable ultimate strength formulas can always be modified without changing the values of specified load factors. Similarly, allowable ultimate strengths for other cases such as shear, bond, bearing, eccentrically loaded columns, and beams reinforced for compression, can be specified to yield a comparable degree of safety,

TABLE 3—ALLOWABLE ULTIMATE STRENGTHS

Case	Type of reinforcement		Tested ultimate strength formula	Allowable ultimate strength formulas*	Tested / Allowable
Beam failure in steel	Structural		$M = K_2\ bd^2$	$M = 0.81\ K_2\ bd^2$	1.23
	Intermediate		$M = K_1\ bd^2$	$M = 0.75\ K_1\ bd^2$	1.33
	Hard		$M = K_3\ bd^2$	$M = 0.60\ K_3\ bd^2$	1.67
Column axial load	Spiral	Concrete	$N = 0.85\ f_c'\ A_g + f_y\ A_s$	$N = 0.34\ f_c'\ A_g + 0.6\ f_y\ A_s$	2.50
		Steel			1.67
	Tied	Concrete	$N = 0.85\ f_c'\ A_g + f_y\ A_s$	$N = 0.27\ f_c'\ A_g + 0.48\ f_y\ A_s$	3.15
		Steel			2.08

*Based on similar margins of safety as provided in the present codes.

when tested formulas are available. For beams whose failure starts in compression of concrete, a greater margin of safety is desirable and lower allowable ultimate strength formulas must accordingly be specified.

PROPOSED LOAD FACTORS

Having fixed the standard for allowable strengths on the one hand, it is now necessary to pick the design loads on the other. After the design loads are specified, we will then be ready to choose proper load factors. In our first attempt to apply ultimate design, it is deemed advisable to keep as much of the present codes as possible, so long as they can be consistent with the proposed method. Hence it seems best to use present specified design loads as they are set up in the respective specifications and only to multiply them by proper load factors for the purpose of ultimate design.

In the choice of proper load factors, it will also be well to keep the following principles in mind as guides.

1. Load factors must be chosen with regard to the maximum anticipated increase in loads. This is a simple, realistic approach.
2. Load factors should be so chosen as to yield total weight of structures comparable to those obtained by conventional methods. In the judgment of many engineers, ultimate design should not be aimed at saving in material during its early stages of application, but rather it should be aimed at the employment of superfluous materials in certain parts to strengthen the weaker links.
3. Load factors should be so chosen as to yield consistent degrees of safety for the different types of loads on different structures. This is a main aim of ultimate design.

With these guiding principles in mind, it is now possible to proceed with the choice of load factors as follows.

Dead load

When choosing the proper values of load factors, let us start with dead load. It is fairly safe to assume that due to variation in density of materials and dimensions of structures, including the effects of adverse distribution, the actual dead load will not in general exceed the design dead load by more than 10 to 15 percent. All things considered, an allowance of 20 percent will likely provide for all uncertainties, including the possibility of future increase in dead load, such as the addition of some surfacing on a bridge or partitions in a building. It is of course possible that future increase may occasionally exceed 20 percent, but it will not be significant, considering the rare possibility of such combinations with simultaneous maximum increase in live and other loads. In view of the fact that ultimate design specifications in other countries generally use a dead load factor of 1.2 and also that this value has been frequently advocated by other authors on ultimate design, this will be recommended for the time being.

One check at this point is the relative safety of such a factor as compared to the conventional method of allowable stresses. Taking intermediate grade steel as the basis, a factor of 1.2 designed against a unit stress of about 30,000 psi means designing the present dead load for a stress of 30,000/1.2 = 25,000 psi. This, when compared to the present allowable of 20,000 psi,

does indicate a less conservative design. But since we are more certain about dead load than any other load, it should be considered justifiable to decrease the margin of safety for dead load, although excessively high dead load stresses are not desirable, in view of resulting deflections especially when considering plastic flow. Since, when rating old structures and when checking for overloads, such stresses as 30,000 psi were used, this factor of 1.2 used in connection with stress in steel of 30,000 psi seems about right.

When dead load and live load are of opposite signs it is recommended that 0.8 of the dead load should be considered in counteracting the live load.

Live load

The next load factor is that for live load. This must be adjusted with respect to the actual possibility of live load increase on the one hand, and with the relative economy of the structure as a whole on the other. Considering future increase, it is safe to say that only in the rarest instances has live load on any structure been increased by more than 100 percent. Data on bridges seem to indicate that the maximum stress due to any prospective vehicle will increase the present design loadings by about 80 percent. While an extraordinary military loading might be heavier, its stress producing effect can usually be minimized by controlling its speed and position. Buildings are seldom subjected to twice their design load. Hence a live load factor of 2 should be considered sufficiently conservative, provided the corresponding stress produced is not excessive. Intermediate grade steel designed at 30,000 psi allowable ultimate strength would have a stress of $30,000/2 = 15,000$ psi under service loads, a more conservative value than the present specifications.

As to economy, it will be shown later that the additional material for live load design would about offset the saving in material for dead load design. This live load factor of 2.0 is primarily intended for bridges and buildings, but is believed to be about right for many other structures as well.

Strain loads

A load factor of 1.5 is suggested for strain loads (such as temperature stresses). This will yield a design stress of about $30,000/1.5 = 20,000$ psi in intermediate grade steel due to the specified strain loads. This stress is just what we have under the present specifications.

Wind load

Similar to strain loads, a load factor of 1.5 is suggested because this will yield a design stress of about $40,000/1.5 = 26,700$ psi in intermediate grade steel due to the specified wind load. This stress also represents no change from the present specifications.

ALTERNATIVE LOAD FACTORS

The above proposed load factors are to be used with a set of allowable ultimate strength formulas which are to be increased by one-third when considering wind. An alternative method of specifying load factors can be

TABLE 4—LOAD FACTORS

Load		Proposed load factors*		Alternate load factors†	
		Without wind	With wind	Without wind	With wind
Dead load	Same sign as other loads	1.2	1.2	1.6	1.2
	Opposite sign to other loads	0.8	0.8	0.8	0.8
Live load		2.0	2.0	2.67	2.0
Strain load		1.5	1.5	2.0	1.5
Wind load		—	1.5	—	1.5

*Allowable ultimate strengths to be increased by ¼ when considering wind.
†Same allowable ultimate strengths whether with or without wind.

adopted if it is desired to use only set of allowable ultimate strength formulas, with no reduction when not considering wind. This method is to specify a higher set of dead, live, and strain load factors when not considering wind. By using a second set of load factors it is possible to obtain exactly the same results as the proposed method. Since the allowable strengths would all be increased by one-third over the proposed method, the load factors should also be increased by one-third, thus raising dead load factor to $1.2 \times 4/3 = 1.6$, live load factor to $2.0 \times 4/3 = 2.67$, and strain load factor to $1.5 \times 4/3 = 2.0$ when not considering wind. These factors would be used together with a basic stress in intermediate grade steel of about 40,000 psi instead of 30,000 psi. For comparison, both the proposed and the alternative sets of load factors are listed in Table 4.

Whether using the proposed or the alternative method, two different sets of calculations must be made, one considering wind and another not. If one set of load factors be specified, then different allowable ultimate strengths must be permitted. If only one set of allowable ultimate strengths is specified, then two sets of load factors must be used. It is the opinion of the writer that the proposed method, using one set of load factors, involves slightly less computation, and is preferable, although there is apparently no serious objection to either method. If the alternative method should be preferred, simpler numbers such as 1.5 for dead load and 2.5 for live load might perhaps be adopted, and some adjustments might be made in the allowable ultimate strengths to maintain the same degree of safety if desired.

It might be well to point out again that in an effort to put ultimate design into practice, we might at this early stage have to tamper with present specifications as little as is consistent with principles of safety and economy. Many questions such as wind load provisions, desired safety for different grades of steel, can be left to future modifications when and if they can be more definitely justified.

TABLE 5—COMPARISON OF APPROXIMATE DESIGN STRESSES*

Item		Present specifications		Proposed or alternate method	
		Without wind	With wind	Without wind	With wind
Design stress with all load factors applied		—	—	30,000	40,000
Design stress without load factors applied	Dead load (same sign as other loads)	20,000	26,700	25,000	33,300
	Live load	20,000	26,700	15,000	20,000
	Strain load	20,000	26,700	20,000	26,700
	Wind load	—	26,700	—	26,700

*Stresses are in psi, for intermediate grade steel in bending.

Table 5 lists a comparison of the approximate design stresses in intermediate grade steel in bending, by both the present specifications and the proposed or alternate method. A glance at this table will reveal that there is practically no change proposed for wind or strain loads. For dead load, the design is less conservative, while for live load, it is more so. In this sense, the factors proposed simply form one version of balanced design. With the adoption of ultimate strength formulas, additional changes will be introduced insofar as such formulas may yield results different from the elastic theories.

COMPARISON OF PROPOSED WITH CONVENTIONAL METHOD

To compare structures designed by the proposed load factors with conventional designs, curves are drawn showing the relative safety and economy of the two methods, again taking intermediate grade steel in bending as an example. With a yield point of 40,000 psi and an allowable stress of 20,000 psi under the present specifications, it has a margin of safety of 1.00. But should dead load increase to $1.2DL$ and live load to $2LL$, the resultant margin of safety would be given by the formula

$$\text{Margin of safety} = 2\,\frac{R+1}{1.2R+2} - 1 = \frac{0.8R}{1.2R+2}$$

Where $R = DL/LL$ ratio

As shown in the curve (Fig. 3a) the margin of safety varies from 0 to 0.67, depending upon R. The proposed design method provides a uniform margin of safety of 0.33 for $1.2\ DD + 2\ LL$, while for $1\ DL + 1\ LL$ the

$$\text{Margin of safety} = \frac{2}{1.5}\,\frac{1.2R+2}{R+1} - 1 = \frac{0.9R+2.5}{1.5R+1.5}$$

and varies from 1.67 to 0.60. Margins for the two methods coincide at the point of $R = 1.67$. For lower DL/LL ratios, the conventional method yields

dangerously low margins of safety, should there be any increase in load. For R greater than 1.67 the proposed method results in saving of material, which can be advantageously used to strengthen the weaker links which exist in parts of low DL/LL ratios.

It could perhaps be questioned whether the proposed method might result in the excessive weakening of longer spans which are the more important members in structural systems. The answer is no. First, while the doubling of live load on short spans is sometimes possible, it can practically never happen on long spans. Hence the use of a uniform live load factor of 2 already provides a higher margin of safety for longer spans. The same is true of the dead load factor of 1.2. Furthermore, since the main portion of stress in long spans is due to dead load, the range of stress as caused by live load is smaller and the number of repetitions is also less. Thus higher safety is still provided in the main carrying members and long spans under the proposed load factors since there is less danger of fatigue failure. This higher safety is probably partly justified in view of the relative significance of such members.

The present double live load clause yields a margin of safety given by the formula

$$\text{Margin of safety} = \frac{2}{1.5}\frac{R+2}{R+1} - 1 = \frac{0.5R + 2.5}{1.5R + 1.5}$$

which is shown by dotted curve in Fig. 3a. It is seen to yield results quite close to the proposed method. This clause, although deemed desirable, is at the present time not applicable to bridge floors or buildings, primarily because of resultant heavier designs. If the proposed method is followed the saving in material from the longer spans can be used to offset the additional materials required for shorter ones.

In Fig. 3b, a comparison of the two methods considering wind load is shown, assuming wind load to be equal to live load stresses, as an example. In this figure a similar effect of the proposed load factors is observed as for Fig. 3a. Also, comparison is shown in Fig. 3c for dead, live, and strain loads, assuming strain loads to be equal to live loads. In both cases it is seen that the proposed method yields uniform safety under overloads, while the conventional method does not.

CONCLUSIONS

While the choice of proper load factors is an extremely important problem which cannot well be decided by the opinion of any single individual, it is deemed desirable that a set of tentative values be proposed for purpose of discussion. In designing, these load factors are to be applied to specified service loads and the products are to be equated to a set of allowable ultimate strengths. It must be further conceded that the problem of safety of structures is a complex one which needs a great deal of study but at the same time defies a purely mathematical approach. Hence the values proposed here

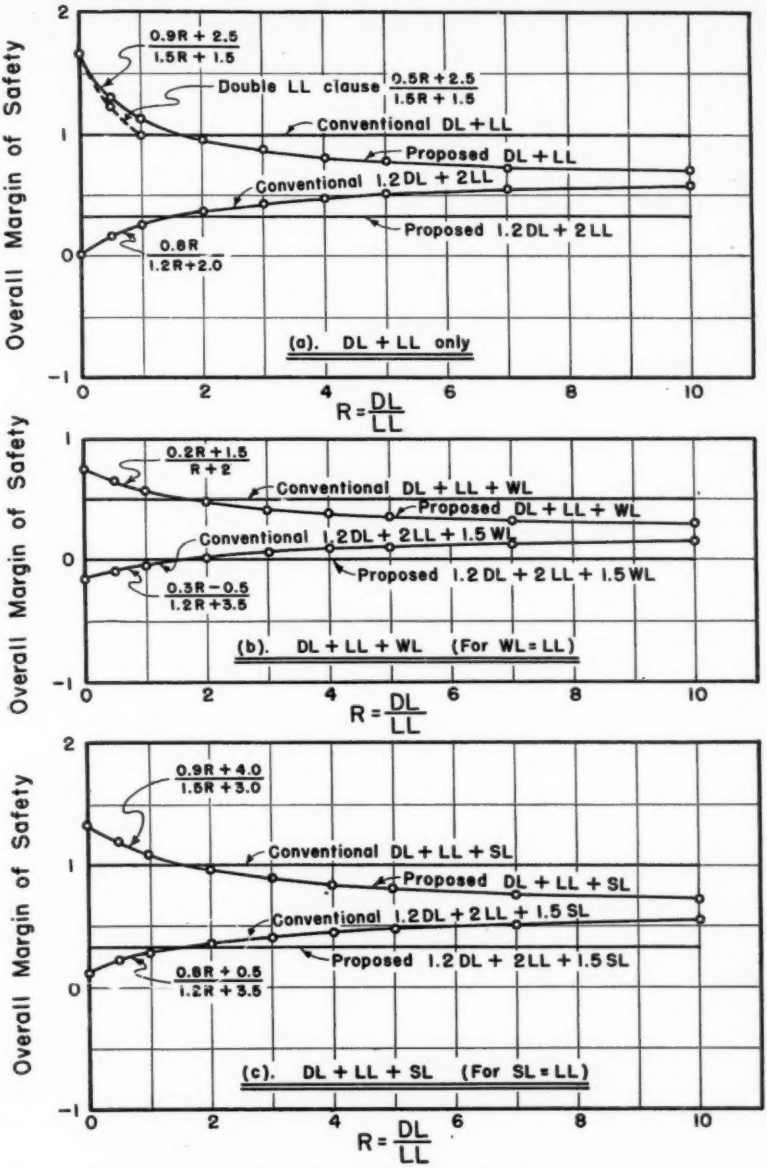

Overall Margin of Safety
0.9R + 2.5 / 1.5R + 1.5
Double LL clause 0.5R + 2.5 / 1.5R + 1.5
Conventional DL + LL
Proposed DL + LL
Conventional 1.2DL + 2LL
Proposed 1.2DL + 2LL
0.8R / 1.2R + 2.0
(a). DL + LL only
R = DL / LL
0.2R + 1.5 / R + 2
Conventional DL + LL + WL
Proposed DL + LL + WL
Conventional 1.2DL + 2LL + 1.5WL
Proposed 1.2DL + 2LL + 1.5WL
0.3R − 0.5 / 1.2R + 3.5
(b). DL + LL + WL (For WL = LL)
0.9R + 4.0 / 1.5R + 3.0
Conventional DL + LL + SL
Proposed DL + LL + SL
Conventional 1.2DL + 2LL + 1.5SL
Proposed 1.2DL + 2LL + 1.5SL
0.8R + 0.5 / 1.2R + 3.5
(c). DL + LL + SL (For SL = LL)

Fig. 3—Comparison of margin of safety—conventional vs proposed methods

may be open to discussion, although the desirability of using load factors on the one hand and allowable ultimate strengths on the other seems rather obvious.

After structures are designed by ultimate strength theories and load factors, they are still to be checked for other items such as fatigue effects and limiting deflections and vibrations. In addition, there are other problems indirectly related to load factors such as the effect of plastic flow, the amount of impact, the application of limit design theory in continuous structures, as well as the possibility of permanent set in certain parts of structures. Some of these cannot be decided upon until after further study, while others will still have to be solved by elastic analysis. Ultimate design cannot be the cure of all evils; but, with the adoption of proper load factors, its application should yield more logical and balanced designs, resulting in safer structures without additional cost.

ACKNOWLEDGMENT

The author is grateful to Stewart Mitchell of the California Division of Highways and Alexander C. Scordelis of the University of California for their helpful comments concerning the preparation of this paper.

Discussion

By RAYMOND ARCHIBALD*

Professor Lin is to be complimented on the brief, interesting, and accurate manner in which he has presented this rather controversial subject. As stated in his closing remarks, its primary purpose is to stimulate discussion and interest to the extent of adoption in some form or another, not necessarily his. It recognizes the defects or faults of our present methods and offers one solution of which there are many.

The application of ultimate design theory considers members stressed into the plastic range. This writer's conception is about the same as outlined in the paper and deals with service loads developing stresses or strains within the elastic range.

This discussion will be confined mostly to precautionary measures that should be considered. It should be understood that the writer is in favor of such a design procedure and has been an advocate of it for the last nine years, as any of the AASHO Bridge Committee members can verify. The method proposed contemplates an overload factor in the neighborhood of 1.2 for dead load and 2.0 for live load. Some of our "empirical formulas" for load distribution may have to be supplemented by more accurate design procedures. On long spans where the dead load stresses are a major portion of

*Chief Western Headquarters, Bureau of Public Roads, San Francisco, Calif.

the total stresses, this stress is acting at all times, therefore is a continuing stress and does not carry a factor of strength for impact or sudden shock that dynamic loads do. It has been shown by tests that sudden shocks can be absorbed to a point beyond a theoretical ultimate sustaining strength.

Lin suggests an increased allowable strength when considering wind. Wind stresses should be calculated as accurately as possible and used in all cases at normal ultimate design stresses. It may not be safe to adhere to the present method of allowing an increase in design stress when considering several combinations of stress-producing forces. The proper approach would be made by ascertaining what portion of the forces or loads will be applied at one time. To illustrate, it is not expected that full live load would be on a highway bridge when the wind force is of such proportions as assumed in the design. However, this might not be the case in a building or a railroad bridge, and in fact, probably just opposite condition exists.

In the use of a dead load factor, care must be exercised in those cases where the dead load might tend to reduce the stress in a member, such as a continuous or cantilever beam.

Many failures, other than foundation failures, can be traced to stress raisers or notch effect caused by poor or faulty details. In some cases, the average stress in a member is well below the present design stress as used now, and failure occurs by tearing or shearing at some point of highly concentrated stresses. With stresses approaching the yield point, more attention must be given to details. It does not necessarily mean that higher unit stresses result under normal loading, but overloads which can be expected to occur occasionally on highway bridges must be anticipated and reckoned with.

We cannot overlook the fact that in the conventional design of a concrete beam, excessive cracking will result from high stresses in the reinforcing steel. In long concrete spans where the dead load is heavy the over-all load factor must not permit high tensile stresses to develop.

The strengths of materials are judged or based on tests of samples. In the case of steel this may be a very small section compared to the total area, and concrete is usually based on a 6 x 12-in. cylinder strength. We must have the proper correlation between the strength of the materials in the member and the test sample.

Consideration should also be given to combat military loads in time of war on highway bridges. Any overload factor for the design live load should insure the safe passage of these vehicles, although not necessarily without some damage to the structure.

We are having rather good luck with our present design of retaining walls which support horizontal earth loads with just enough failure to indicate a rather close design procedure. It may be necessary to look into this rather extensively when adopting the ultimate design for this type of structure.

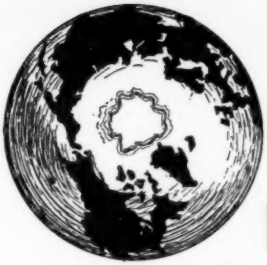
Ultimate design is not based on any premise to save material. In most cases it merely reduces the amount of material at one portion of the structure

where the DL/LL ratio is large and places it in another portion where this ratio is small. It is an attempt to make all portions of a structure and all types of structures designed for similar conditions capable of sustaining the same live load factor without injury, which makes sense.

As mentioned in Professor Lin's paper, the ideal cannot be accomplished because a structure will have a "balanced design" only for the conditions of the design load used but an approach should be made to attain this ideal by this method.

The question might rightly be asked: shall we design our structures with a balanced condition for an adequate design load of today and have an unbalanced condition when overloaded, or should we design them for an unbalanced condition today and a balanced condition when overloaded. Personally, this writer favors the latter where the overloads produce a safe and balanced design. The selection of the load of the future is very important in ultimate design procedure. Our present designs are examples of structures designed for the same loading conditions capable of carrying many times the design live load in some instances and less than two times in others. That even applies to different parts of the same structure.

It might even be that we can pursue this problem of live load factors on a bridge by first designing the structure using design stresses recommended for ultimate design, with the heaviest military loads that might be expected to use the bridge during actual combat. Then check the design against a nominal design load that would be considered a legal load. In the latter case, no stresses should be tolerated which would be considered excessive under many load repetitions. In this way we could take care of the infrequent permitted overload and still provide for the normal or legal load at stresses considered safe under fatigue conditions.



Letters from Readers*

BY WAY OF SYNOPSIS

T. F. WILLIS compares techniques for measuring air in hardened concrete.

C. F. MOHR discusses the possibility of designing specific cement types to give maximum satisfaction with each general type aggregate.

The new CRSI design manual is an important step toward standardization of design methods long awaited by LOUIS A. WARNER.

Measuring Air in Hardened Concrete (LR 48-28)

Two points mentioned in the Discussion† of the paper by Brown and Pierson "Linear Traverse Technique for Measurement of Air in Hardened Concrete"‡ deserves further comment. First is a statement in the authors' closure, relating to the disclosures of the statistical analysis of their data, which follows: "What the work has shown is (1) that a 6 x 8-in. surface may be considered a representative sample of a normal concrete with 1½-in. maximum aggregate, and (2) that 100 in. of traverse covering this surface determines the air content of the sample within a standard error of 0.4 percent." I believe a more precise wording would have been "What the work has shown is (1) that *for a cylindrical specimen* of ordinary concrete, *6 in. in diameter by 8 in. long* containing aggregate of 1½-in. maximum size, a section 6 x 8 in. in area *cut so as to include the axis of the cylinder*, may be considered a representative sample of the specimen; and (2) that *eleven or twelve traverse lines, each extending over the 8-in. width of the section, located at uniform intervals over the 6-in. dimension of the section*, determines the air content of the section within a standard error of ± 0.4 percent."

The significance of my proposed rewording of the statement by Brown and Pierson is (1) that the data in the paper yield no information on the possible sampling error if one used a 6 x 8-in. section to represent a mass of concrete *larger* than the 6 x 8-in. cylinder; and (2) that the last part of the authors' statement might be subject to an unwarranted interpretation. Concerning the last portion of the statement, it is perfectly correct for them to conclude from their statistical analysis that 86 in. of traverse should yield a mean with a standard error of ± 0.4; but it should be emphasized that these 86 in. are to consist of the required number of continuous traverses, each 8 in. long. Without this qualification one might assume that the mean of 86 random 1-in. traverses would also have a standard error of ± 0.4 percent air. This could, but would not necessarily, be true. One can see intuitively that the larger the size of coarse aggregate particles, the greater the dispersion of the results of short random traverses.

A little emphasis on the significance of the term "Standard Error" might also be of value. As applied to the data of Brown and Pierson it is simply this: one may state

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†ACI JOURNAL, Part 2, Dec. 1951, *Proc.* V. 47, pp. 124-1.

‡ACI JOURNAL, Oct. 1950, *Proc.* V. 47, p. 117.

TABLE 1—RELATIVE PRECISION IN MEASURING AIR CONTENT

Section from core No.	No. of 8 in. traverses	Mean air content	Standard deviation	Standard error	Confidence limits	
					80 percent	90 percent
3	12	3.05	.5552	±0.16	±0.23	±0.30
4	12	3.89	.6197	±0.18	±0.26	±0.34
5	12	4.27	.9973	±0.29	±0.41	±0.54

that the "true"* air content of the section lies in the interval $A \pm 0.4$, (where A = the percent air calculated as the mean of eleven 8-in. traverses) with approximately a 65 percent probability of being correct; and conversely there is a 35 percent probability of the statement being wrong.

Probably most analysts of hardened concrete would be satisfied if they were fairly certain that an estimate of the percentage air content was within ±0.4; but a statement to this effect, when there is a 1:3 chance of being wrong, cannot be made with a very definite feeling of certainty. If the statement could be made with an 80 percent probability of being correct (*i.e.*, with only a 1:5 chance for being wrong) most of us would be more satisfied. The data of Brown and Pierson furnish information for determining the number of traverses necessary to achieve this. Consideration of items 1, 2a, 2b, and 3c, of Table 2 of the original paper indicates that the standard deviation of the mean is $\sigma = 1.30$. With this as a starting point and using certain well established statistical relationships, it can be calculated that about twenty 8-in. traverses would be required to obtain 80 percent confidence limits of $A \pm 0.4$. A similar analysis discloses that 80 percent confidence limits of $A \pm 0.5$ can be attained with about fourteen 8-in. traverses, 90 percent confidence limits of $A \pm 0.5$ would require twenty-one 8-in. traverses, and twelve 8-in. traverses would give 90 percent confidence limits of $A \pm 0.7$.

In our use of the linear traverse method on 6 x 8-in. sections of pavement concrete with 2-in. maximum size aggregate, we have found the standard deviation of the calculated mean of a section to be somewhat smaller than that indicated by the Brown and Pierson data. This of course results in a smaller standard error and narrows the confidence limits for variable probability levels, and indicates the possibility of attaining somewhat greater precision for the calculated mean air content. Verification of the indication could only be achieved through replicate analyses by two or more laboratories and operators. Table 1 shows a few typical results.

The second point is Mrs. Mather's suggested use of the "point-count" method, which was called to my attention by Dr. Chayes.† Some time ago we had occasion to check the air content of a stretch of pavement suspected of being over-aerated. Having no experience by which we could determine the sampling error involved in using cores to represent a two cubic yard batch of concrete, a rather large number of cores (6 in. in diameter by 8 in. long) were drilled and sectioned. The prospective work involved in testing all these by the linear traverse method caused us to experiment with the "point-count" method. Results were very gratifying.

Two specimens were analyzed (one section from each specimen) by both methods, and the results subjected to statistical analysis. One specimen was a 3½ x 4½ x 16-in. laboratory prism sectioned equidistant from the two 4½ x 16-in. faces. Air content of the concrete, while in the plastic state in the prism mold, was determined in a special air meter. The other specimen was a 6-in. diameter core 8 in. long, drilled from a pavement and sectioned through the axis of the core. See Table 2 for results.

From these results it would appear that a few more traverses of points would have made the "point count" analysis fully as precise as the linear traverse. The actual operating time required by the microscopist

*In reality this is the so-called "objective mean" that would be approached as a statistical limit when the number of traverses was increased indefinitely.

†Private communication from Dr. Felix Chayes, May 1950.

TABLE 2—COMPARISON OF LINEAR TRAVERSE AND POINT COUNT METHODS

Spec. No.	Method of analysis	No. of traverses	Mean air content	Confidence limits	
				80 percent	90 percent
Beam 1 AB 3 Meter air cont. = 4.98 percent	Linear traverse*	7 of 16 in.	4.76	±0.36	±0.48
	Point count†	7 of 16 in.	4.91	±0.60	±0.80
Core S-3	Linear traverse‡	12 of 8 in.	9.14	±0.50	±0.66
	Point count§	12 of 8 in.	9.39	±0.62	±0.82

(*) Traverses spaced at 0.64 in.; equivalent to 1.5 in. of traverse per sq in. of section.
(†) Traverses spaced at 0.64 in.; points of traverse spaced at 0.0625 in.; equivalent to 1 point for each 0.04 sq in. of section.
‡Traverses spaced at 0.5 in.; equivalent to 2 in. of traverse per sq in. of section.
(§) Traverses spaced at 0.5 in.; points of traverse spaced at 0.0625 in.; equivalent to 1 point for each 0.03 sq in. of section.

on a specimen with 5 percent air is about 50 minutes for twelve 8 in. point traverses, while about 2½ hours are required for twelve 8-in. linear traverses. Considering that eye fatigue is much less for the point count method, the difference in total elapsed time for the two methods of analyzing a specimen gives the point count method an even greater advantage than is indicated by the above figures. Therefore, even with the additional point traverses, required to give a precision equivalent to that of the linear traverse, the point count method will still save about 50 percent of the time required for the linear traverse analysis.

For the "point count" method we use the same apparatus described in my discussion of the Brown and Pierson paper. The only modification is the addition of a wheel fixed to the right hand end of the lathe lead screw. This wheel has two protruding knobs at the midpoints of the circumference which engage a fixed spring and give an audible click as the wheel revolves past the point of contact. The operator merely watches the specimen through the microscope and observes, and records on punch counters, whether or not the cross-hair intersection falls within the confines of an air bubble section at the time he hears each click. Since our lathe has a "quick change" mechanism we can readily adapt the speed of travel to the requirements imposed by the number of bubble sections encountered. As Mrs. Mather points out, by using additional counters, estimates of the coarse and fine aggregate contents can be made simultaneously with the air-content determination.

All things considered, when air content is the only information about the air bubble system desired, the "point count" method is by far the best analytical method we have used. It is much faster than the linear traverse and less fatiguing to the operator. Its deficiency, as pointed out by Brown and Pierson, is its inability to yield information about bubble size and spacing.

T. F. Willis, Chief, Research Division, Missouri State Highway Dept., Jefferson City, Mo.

Why Not a Particular Type Cement for each General Type Aggregate? (LR 48-30)

In connection with the review by Donald M. Agrimson, page 111, September 1951 issue, of "Comments on Imaginative Cement and Concrete Chemistry," the following thoughts are considered pertinent.

Why not go one step further in the logic of a policy for portland cement manufacturers to modify cements to function best with aggregates whose chemical and physical properties are fixed, which seems to be good engineering, and select aggregates on the basis of properties essential to produce the most serviceable concrete.

Limestone is widely distributed, and the most uniform type could be chosen for building construction where resistance to weather-

ing, wear, and slipperiness are not essential but fire resistance and high strength are more important. For building construction uniform high grade slag would have the additional advantage of light weight.

Probably the largest single field of concrete construction is for paving. The essential properties for a concrete highway are durability, high flexural strength, resistance to wear, nonskid surface, and low thermal and moisture volume changes.

Concrete with these desirable properties can be produced with aggregate consisting of basic igneous rock, basalt, diabase, and gabbro. They are widely distributed over a large area in the United States. For instance, in the New England States, New York, New Jersey, Pennsylvania, Maryland, and Virginia this trap rock is available in unlimited quantities but is not more widely produced because it costs more to process than softer and less durable rocks.

If it is good economics for the railroads in such an area to pay a premium for this trap rock as railroad track ballast because of high resistance to crushing, wear, and weathering for main line heavy traffic and to use the cheaper but less durable rock for branch lines, such a policy would also be good engineering in highway construction.

We need a nation-wide impartial, noncommercial, concrete research organization with trained, experienced engineers with the necessary research equipment to make these studies, geologists to classify the various rocks, and chemists to design cement to function best with the selected aggregates.

C. F. Mohr, Sales Promotion Engineer, Kingston Trap Rock Co., Kingston, N. J.

Why Complicate R/C Design? (LR 48-31)

Have you seen the new CRSI Design Manual?* It is a good handbook and will simplify the planning and estimating work of the design office. Although the publication is not without flaws, it fills an important gap with a comprehensive compilation of tables and information.

Specifically, the tables for eccentrically loaded columns, which incidently don't need any page hopping to use, and the standard stirrup numbering system are the best. Collecting the tables under one cover saves a good deal of time and energy in looking for miscellaneous references.

All the necessary answers are there and quite a few extra. The standardization theme is a step in the right direction, but needs expanding. The tables are too complete and cause unnecessary confusion. If every third line in the tables was omitted the designers work would be easier and the structure wouldn't cost any more. The various inherent uncertainties as to the loads, moments, materials, workmanship, and costs hardly justify the large number of solutions therein and probably makes the designers choice that much more difficult.

Present concrete design practice provides an infinitely large number of sections that will satisfy the conditions of loads and assumed stresses. Eliminating odd dimensions and sizes of members will simplify the design, the details, and field work required.

Steel and timber interests have used this method of standardization and simplification to great advantage, and have eliminated the less common sections. Concrete designers can do the same and profit well by it.

Some mathematically inclined or academic minded engineers will reply that every structure should be individually computed from start to finish. All this is fine, but we have an obligation to our clients somewhat akin to that of an accountant. It is far better that he is able to keep track of the dollars, than waste his time trying to balance the pennies and inadvertently overlook the bigger questions.

Louis A. Warner, New Haven, Conn.

*Published by Concrete Reinforcing Steel Institute, 38 S. Dearborn St., Chicago, Ill. 1952, $5.00.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Influence lines by corrections to an assumed shape

JAMES P. MICHALOS and EDWARD N. WILSON, Separate No. 124, V. 78, *Proceedings*, ASCE, Apr. 1952. 15 pp. $0.50

An analytical procedure for obtaining influence lines for continuous structures by successive numerical corrections to an assumed shape. The structure may have any variation in cross section and may or may not be subject to sidesway. Any degree of precision is possible.

Flow and creep in composite beams (Schwinden und Kriechen bei Verbundtraegern)

MARIA ESSLINGER, *Der Bauingenieur* (Berlin), Jan. 1952, V. 27, No. 1, pp. 20-26

Reviewed by ARON L. MIRSKY

Analysis of effect of flow and creep on composite beams (concrete slab on steel stringers) with especial reference to beams subjected to negative bending moment and to beams with redundant supports (continuous beams). Author derives formulas for various cases; numerical examples are also included.

Design and construction of the new deep-water quays at Leith

MATTHEW COCHRANE WHITE, *Proceedings*, Institution of Civil Engineers (London), Part II, Feb. 1952, V. 1, No. 1, pp. 55-96 (including discussion)

Reviewed by ARON L. MIRSKY

These quays, 500, 424, and 120 ft in length, were constructed of 69 concrete caissons, precast adjacent to their final locations and floated into place. The concrete adjacent to the steel cutting edge was prestressed. Final excavation for seating the caissons was done in the dry, under compressed air.

Construction practices on the double locks of the Lower Isar (Die Bauausfuhrung bei den Staustufen der Unteren Isar)

J. STOCKER, *Der Bauingenieur* (Berlin), Jan. 1952, V. 27, No. 1, pp. 26-33

Reviewed by ARON L. MIRSKY

This project, undertaken to aid in meeting the power shortage in Bavaria, contains 9 almost identical reaches and dams in a river length of 73 km. This article describes the construction of the first section: power house, spillways, high-water control dams, etc. Of special interest is the high degree of mechanization of the construction operations, including concrete placement.

Frost action in roads and airfields

A. W. JOHNSON, *Special Report* No. 1, Highway Research Board, 287 pp.

Pertinent findings and conclusions from a great number of articles on the effect of climate, soil, ground water, pavement, and traffic on frost action due to freezing and thawing. Various theories on causes of pavement disruption are summarized and corrective measures are described. Major emphasis is on problems in temperature climates although some attention is given to conditions in permafrost areas. An extensive bibliography covering literature from 1765 to 1951 is appended.

Prestressed concrete beams in the roof construction of the Kaukopaa cardboard mill (in Finnish)

SIMO SCHULTZ, *Rakennusinsinoori* (Helsinki), V. 7, No. 4, 1951, pp. 42-51

When planning the roof construction of the 312-m long factory building it was

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decided to use prestressed concrete beams, after the different alternatives were considered. Construction methods are described and the experience gained in the use of prestressed concrete is discussed. Prestressed concrete was found, in this case, to be far more economical than ordinary reinforced concrete and also permitted a better organization of the work at some stages.

Influence line tables

GUSTAV GRIOT (Translated and enlarged by HAROLD G. LORSCH), Frederick Ungar Publishing Co., New York. 87 pp. $3.75

The four-place influence line tables greatly reduce the calculations required for the analysis of continuous beams under dead and live loads. They may also be used to advantage with Kleinlogel's *Rigid Frame Formulas*. All tabulated values for moments were computed analytically by two independent methods and checked by large-scale graphs. Although the fourth decimal, while accurate in many cases, should be regarded as approximate only, no more is usually required for most computations.

Data book for engineers V. I—Design. V. II—Specifications and cost

E. E. SEELYE, 2nd Edition, John Wiley & Sons, Inc., New York. V. I, 658 pp., $10.00. V. II, 505 pp., $13.00

To revised V. I has been added data on design of spread footings, airfield layout and design, and rigid frames. Other new material includes standards for swimming pools, details of gates, flashboards, and fish ladders for dams, and sight distances, maximum curvature, and grades for expressways and turnpikes.

Major additions to V. II include specifications for the elements of an industrial building, a sample evaluation report, swimming pool specifications, information on charging off obsolescence, costs of miscellaneous structures such as deep sewers and tunnels, costs for the elements of buildings, and square foot costs of industrial buildings.

Progress of construction of prestressed concrete bridges in Nord and Pas-de-Calais (in French)

F. DUMAS, *Travaux*, No. 204, 207, 208, and 209, Oct. 1951, Jan., Feb., and Mar. 1952.

Reviewed by M. W. JACKSON

This series of articles reports details of construction of several bridges, with numerous photographs and drawings. Laboratory tests of the steel and concrete used in the bridges are reported. The final two issues describe the bridge of the Hippodrome at Lille, completed in 1951. This bridge is approximately 225 ft long, 40 ft wide, and is placed on a slight skew. The bridge is supported by 10 I-shaped girders with depths of approximately 10 ft. The steel in the girders is post-tensioned.

Tests of air-entraining admixtures

Technical Memorandum No. 31 6-237, Waterways Experiment Station, Vicksburg, Aug. 1951. 24 pp. $0.75

Brief discussion of the procedures for testing air-entraining admixtures and the results of tests on 75 samples representing 15 brands or types of materials manufactured by 10 different manufacturers. The test method places limitations on bleeding, compressive strength, flexural strength, resistance to freezing and thawing, and bond to steel although the latter is not reported upon. Of those agents which failed to comply with the criteria, inability to develop the required compressive strength in the specified period was the principal cause.

Deflections in slabs on elastic foundations

WALTER W. EWELL, and SHIGEO OKUBO, *Proceedings*, Highway Research Board, V. 30, 1950, pp. 125-133

The fundamentals of a new method of slab analysis are presented. This analysis, employing a simple extension of moment distribution principles, has given results that have been substantiated by the theories of elasticity and methods of finite differences.

The technique consists of a transformation of any given slab into a gridwork of beams, a systematic load deformation, and a subsequent distribution of bending moment and torsional effects on the grid. Although analyses of slabs with grid systems are not new, a uniqueness is realized in this analogy by assuming rigid and structurally indeterminate grid joints throughout the system.

Deflections developed in slabs on elastic foundations are investigated. The theory is completely illustrated by the analysis of a slab on a soil foundation.

Placing concrete during frost (Das betonieren bei frost)

F. BOHM and W. DRECHSEL, Wilhelm Ernst & Sohn, Berlin, 1951, 79 pp., 7.50 DM.

Reviewed by RUDOLPH FISCHL

In 1928 the first attempt was made to compile the knowledge and experiences with

concrete construction in winter. This enlarged and revised 5th edition discusses all the factors important for winter work, gives details about protection of the aggregates and the fresh concrete against loss of heat as well as protection of the whole structure or special parts of it. The book shows also the necessary special organization and considers the economical importance of this kind of work for German labor conditions.

Los Angeles constructs 2000-car underground garage

CHARLES A. MCMAHON, *Civil Engineering*, V. 21, No. 12, Dec. 1951, pp. 25-30

Reviewed by ROBERT E. WILDE

Unique design features and unusual construction methods were used in the three-story underground parking garage built in downtown Los Angeles.

The garage incorporates the latest thinking in the structural design of concrete slabs. The floor system is a "haunched slab" supported by 26-in. diameter columns spaced 27 ft on centers both ways. On the underside, the slab slopes in four directions from the columns to the one-third points of the spans. These square tapered column capitals are said to be more economical to construct than conventional design of drop panels and column capitals.

The flat slab was analyzed by using the plastic model method (Presan analysis). New-type deformed bars were used for reinforcement, following the ACI Building Code.

Concrete for columns and floor slabs was designed for a compressive strength of 3000 psi; the roof concrete, 4000 psi. About 50,000 cu yd of transit-mixed concrete went into the structure.

Unusual shoring methods were used in excavation; 208 thirty-ft precast concrete columns were set on 9-ft centers around the outside of the excavation. The columns were set by crane, centered, concreted in, and backfilled. These columns later became a permanent part of the exterior wall of the garage.

Since the structure was designed to fit the area contour, there are no level slabs within the main garage area. Construction was further complicated by warped slabs, curved walls, haunches and curved beams.

Gravel compaction and testing, and concrete mix design at London Airport

HAROLD SMITH, *Proceedings*, Institution of Civil Engineers (London), Part II, Feb. 1952, V. 1, No. 1, pp. 1-54 (including discussion)

Reviewed by ARON L. MIRSKY

Author draws a parallel between soils and concrete, and designs concrete mixes on basis of an "optimum moisture content." Claim is made that "there is only one water-cement ratio for a concrete mix and that the generally accepted rule that the water-cement ratio governs the strength appears to follow a definite law only when using 'optimum water'." Further claim is made that the method of placing the concrete is the main factor controlling the design of a mix; that the use of a slump is fundamentally wrong; and that workability is controlled by grading and shape of aggregate and not by addition of water.

Author also compares batching by volume and by weight, somewhat to the detriment of the latter.

Article contains considerable controversial material, as evidenced by the extended discussion (pp. 38-54). The paper is difficult to abstract; reviewer is of the opinion that it is well worth reading.

Studies of compressive stress distribution in simply reinforced concrete near the point of failure

LESTER A. HERR and LOUIS E. VANDEGRIFT, *Proceedings*, Highway Research Board, V. 30, 1950, pp. 114-125

Describes pilot studies using a 6 x 6 x 0.66-in. piece of special optical glass in the compressive area of a 6 x 12-in. x 15-ft heavily reinforced concrete beam resting on supports 12 ft apart and loaded at the third points. The pressure from the concrete to the glass causes strain in the glass proportional to the pressure. Photoelastic photographs gave information from which may be determined the average compressive stress at any distance from the neutral axis. The stress distribution thus found varies with the moisture variation in the beam and with the variation in quality of the concrete at different depths in the beam. In general, the stress distribution may, but seldom does, follow a straight-line variation. The compressive stress in the extreme fiber is usually several hundred psi less than the maximum compressive stress. Indications are that vari-

ations in moisture and humidity at the different layers greatly affect the stress distribution.

Stability problems in prestressed concrete (in Polish)

S. KAUFMAN, *Inzynieria i Budownictwo* (Warsaw), V. 7, 1950, pp. 228-236
APPLIED MECHANICS REVIEWS Feb. 1952 (Olszak)

Expressions are derived for the critical prestressing force by the Ritz-Timoshenko energy method for the general case of cable contacting concrete in many points as well as for continuous contact. Constant and variable cable eccentricity is taken into account. For the single asymmetrically situated contact, the critical prestressing force is reached when one (of the two) parts of the deformed axis remains straight. Thus, one of Magnel's four experiments (the "bizarre" one) is explained theoretically.

Variation of wind velocity and gusts with height

R. H. SHERLOCK, Separate No. 126, V. 78, *Proceedings*, ASCE, Apr. 1952. 26 pp. $0.50

Flow of air in level, open country is adopted as the standard of reference, assuming that the influence of local shielding and unusual topography will be evaluated by the designer in each individual case. Recommendations are based on velocity pressures rather than on design pressures.

The theory of the variation of wind velocity with height, based on the Ekman spiral, is first discussed. The records of a particular storm are used to validate the theory and to provide detailed information regarding gust characteristics. The duration and extent of the minimum effective gust is discussed. A recommendation is made for the variation of velocity pressure with height, to be used in obtaining the design pressure at any height. The magnitude of the design pressure will depend upon the recommended velocity pressure near the ground in each particular geographic location. A special recommendation is made for the case of guyed towers.

Construction of a prestressed concrete road at Crawley New Town

A. J. W. MCINTOSH and JACK MERCER, *Proceedings*, Institution of Civil Engineers (London), Part II, Feb. 1952, V. 1, No. 1, pp. 220-234
Reviewed by ARON L. MIRSKY

This experimental slab was 6 in. thick, 404 ft long and 24 ft wide, with a longitudinal construction joint. Prestressing was by cables 82 ft long arranged in a diamond pattern at 7 ft 6 in. centers; cable anchorages were separate from the slab proper.

Measurement of slab movement indicated that special expansion joints were not needed at the ends of prestressed slabs of this length. However, the formation of contraction cracks (which closed up on the application of prestress) indicated the desirability of using construction joints at 50 ft spacings to prevent random cracking.

Cost was comparable to that of ordinary slab construction.

It should be noted the possibility of having to cut through the slab later was not a problem in this case, since all utilities were placed outside the roadway limits.

Studies of slab and beam highway bridges. Part III—Small-scale tests of shear connectors and composite T-beams

C. P. SIESS, I. M. VIEST and N. M. NEWMARK, *Bulletin* 396, University of Illinois Engineering Experiment Station, Feb. 1952, 136 pp.
Reviewed by IVAN M. VIEST

Tests of one-quarter scale shear connectors for composite concrete and steel bridges are reported. Three types of tests were made as follows: (1) static tests of 64 push-out specimens, (2) static tests of three composite T-beams, and (3) 85 fatigue tests of composite T-beams. The push-out specimens were made with various types of rigid and flexible connectors; the T-beams were made with channel shear connectors only. Strength of concrete, types and dimensions of shear connectors, and type of loading were the principal variables.

Bulletin describes the specimens and testing procedures, and gives the test data in considerable detail. Most of the conclusions are only qualitative. The authors express the opinion that quantitative data should be obtained from tests of full size specimens.

A theoretical analysis of composite beams with incomplete interaction is given in an appendix.

Compound beam with concrete plate prestressed by tie steel (in German)

B. FRITZ, *Stahlbau*, V. 20, No. 8, Aug. 1951, pp. 97-103
APPLIED MECHANICS REVIEWS Mar. 1952 (Massonnet)

Paper follows another by the same author

(*Bautechnik*, 37-42, 1950) which should be read first. The first paper gave an approximate method for calculating the stresses in statically determinate compound beams (used in bridges with concrete slab and steel I-beams) by taking into account the shrinkage and creep of the concrete. The theory was based on certain simplifying assumptions concerning the evolution with time of the residual stresses due to shrinkage and creep. The numerical results were in good agreement with those given by a rigorous analysis (*cf.* Frohlich, *Bauingenieur*, p. 300, 1949) and, in the reviewer's opinion, they are largely sufficient, owing to the uncertainty affecting the law of creep.

In present paper, author extends the basic calculations to a steel-concrete section, where the concrete is prestressed by steel wires. It is assumed that the concrete slab is rigidly connected with the steel beams and that the concrete is prestressed, both at the same time t_o.

Formulas are given for the stresses at this moment, as well as at the time t_e, where shrinkage and creep have practically stopped. These formulas are the basis for the calculation of statically indeterminate systems such as continuous beams and arches tied by a prestressed concrete floor.

Paper treats completely the case of a symmetrical bridge continuous over three spans and prestressed in two zones where the bending moments due to the loads are negative. Obtained formulas are applied to a 540-ft bridge over the Ruhr river.

Contraction joints in concrete dams (La conception des joints de contraction et la construction des ouvrages en beton massif)

A. Lefebvre, *La Technique Moderne-Construction* (Paris), V. 6, No. 12, Dec. 1951, pp. 404-407
Reviewed by Alexander M. Turitzin

As of now, it has been impossible to manufacture a low priced cement that does not generate heat in setting and shrink objectionably during the process of hardening, thereby eliminating the necessity of construction joints in massive concrete structures. The general practice is to divide a massive concrete structure into blocks separated by construction joints to allow for the variation of volume in the mass of concrete. In the case of a dam, it is necessary to subdivide it into blocks along its length so that no shrinkage cracks appear in the concrete mass of each block. In many structures, construction joints play the role of expansion and contraction joints. In structures where the construction joints act as contraction joints only, it is necessary to seal the joints to provide watertightness and to make the structure monolithic.

It is possible to eliminate vertical joints by placing the entire structure in full-length horizontal increments, if vertical holes are provided. These vertical holes, drilled while the concrete is green, are connected to horizontal tubes of sheet metal or plastic placed in concrete, thus forming a grid system of ventilating ducts. It was found that mass concrete, 24 hours old, could easily be penetrated with standard drilling equipment supported directly on the concrete. Construction by full-length placement was found to be more rapid than the general method of concreting separate blocks. However, economy is achieved only in large structures more than 3 m thick.

Acoustic testing of concrete (Acoustisch onderzoek van cement en beton)

P. C. Kreyger, *Cement* (Amsterdam), No. 11-12, Nov. 1951, pp. 202-207
Reviewed by J. W. T. Van Erp

Development and advantages of this non-destructive testing method are enumerated, such as (1) more than one test can be taken on the same part of the structure and reliable averages obtained, (2) consecutive tests on the same spot indicate what changes are taking place in such quantities as the modulus of elasticity during aging, (3) tests can be taken on arbitrary shapes and parts of the structure, apparent defective spots can be tested, (4) influence of such factors as humidity, temperature, and freezing can be investigated on the same piece of concrete at different times, and (5) tests are fast and data are available immediately.

Although the method actually gives only a direct indication of the modulus of elasticity, the strength of the tested concrete may be surmised from the correlation between these two quantities. The dynamic modulus of elasticity is obtained from the tests as against the static modulus of elasticity obtained from conventional load tests.

The dynamic modulus of elasticity is constant at different stress levels, while the static modulus of elasticity decreases at higher stresses and also with slower application of load. Under instantaneous loading, which is practically unobtainable, the static modulus is higher and becomes equal to the dynamic modulus.

Vibrating concrete, with small amplitudes of stress variation, behaves according to the dynamic modulus of elasticity. The definite relation between the dynamic and static modulus of elasticity permits determining the latter by acoustic testing. This is mostly done by the determination of the resonance frequency or by measuring the speed of propagation of longitudinal vibration, as between both of these and the modulus of elasticity a mathematical relation can be shown to exist. Measurement of the speed of propagation of vibration lends itself to tests on actual structures.

Various research projects in this country are mentioned and the methods developed are explained.

Predetermination of water requirement and optimum grading of concrete

NIELS M. PLUM, *Building Research Studies* No. 3, The Danish National Institute of Building Research (Copenhagen), 1950, 96 pp.

From AUTHOR'S SUMMARY

The work reported used combined concrete gradings, *i.e.*, gradings including aggregate as well as cement, and their superiority to normal aggregate gradings is demonstrated.

Traditional grading formulas for percentage grading do not cover gradings with minimum particle size greater than zero. A new formula covering this case is presented and the grading percentage which will give maximum strength, density, economy with regard to strength, and economy with regard to density is determined by tests.

It is further demonstrated that percentage gradings with minimum particle size equal to zero in a double logarithmic plotting will give straight-line grading curves, and that percentage gradings with minimum size greater than zero in the same system will give straight lines when the amount of the single fractions is plotted against the sieve sizes. Since the specific surface is the dominant factor the geometric mean should be used as the mean fraction size.

The seldom recognized fact that the steepness of the optimum grading curve increases with decreasing maximum particle size is quantitatively proved. At the same time it is demonstrated that with increasing maximum particle size, and increasing grading percentage the concrete quality becomes more and more sensitive to inaccuracies in the composition.

A new formula is presented for the calculation of basic water content which takes into account not only the specific surface of the solids but also the steepness of the grading curve. It is also shown that there is no definite relation between the volume of voids in the dry solids and the volume of voids in the concrete.

An extension of the work of Caquot and Faury on molding resistance showed that maximum density is obtained for the combined molding resistance, $CMR = 1$. For unreinforced concrete sections this means that the maximum particle size of the coarse aggregate should not exceed one-fifth of the width of the form.

All concrete warehouse with prestressed concrete roof domes (Reconstruction en beton precontraint des hangars a cotton du Port du Havre)

La Technique Moderne-Construction (Paris), V. 6, No. 12, Dec. 1951, pp. 409-412

Reviewed by ALEXANDER M. TURITZIN

The handling of large quantities of bales of cotton, their short storage period, and the danger of fire requires a large unobstructed area in a well-aerated fireproof one-story structure. This warehouse is 500 m long and 113 m wide topped with curved domes forming rectangular bases, 19.33 x 18.79 m and resting on prestressed reinforced concrete girders supported on cylindrical columns 80 cm in diameter. At the top of each dome are 6.53 x 5.99-m rectangular skylights. The skylights are made of thermolux glass to prevent the effect of a magnifying lens and thereby eliminate the danger of fire which may occur in a cotton warehouse with ordinary glass windows. Four glass panels, more brittle than the others, are located at the center of the skylight, designed to provide

escape for hot gases and admit fresh air, thus eliminating the rapid spreading of fire.

Tie rods made of 18 wires 5 mm. in diameter are placed in girders supporting the domes. There are four tie rods per dome. If the tie rods were exposed, they would be vulnerable to fire, and their elongation under heat might cause the collapse of the domes themselves. Freyssinet's method of conical end anchorage was used and the tie rods were not bonded to concrete. An important point to consider in the use of the prestressed tie rods is their effect on the watertightness of the roof domes. By prestressing the tie rods, it was possible to eliminate most of the tensile forces in the domes, thus realizing watertightness. Wood forms for six domes were built simultaneously on the ground. They were raised in place, and after the concrete hardened and the tie rods were prestressed, they were lowered, moved and raised into the new position. It was possible to re-use the same wood forms as much as ten times. The construction of this warehouse opens a new vista for the application of prestressed concrete to a problem which, until now, has seemed to be practically without a solution.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirability of publishing a list of those periodicals being regularly scanned for articles dealing with the field of concrete. Included also are the publishers' addresses for convenience should correspondence with them be desired.

Annales de L'Institut Technique du Batiment et des Travaux	28 Boulevard Raspail, Paris 7, France
Bauingenieur	Reichpietschufer, 20, Berlin, W35, Germany
Bautecknik, Die	Hohenzollerndam 169, Berlin-Wilmersdorf, Germany
Betong	Odengatan 41, Stockholm, Sweden
Beton og Jernbeton	16 Godthaabshave, 3rd Floor, Copenhagen C, Denmark
Beton-und Stahlbetonbau	Hohenzollerndamm, 169, Berlin-Wilmersdorf, Germany
Beton-Teknik	Kalvebod Brygge 2, Copenhagen V, Denmark
Betonstein Zeitung	Frankfurter Strasse 5, Wiesbaden, Frankfurt Am Main, Germany
Bulletin, ASTM	1916 Race St., Philadelphia 3, Pa.
Bulletin, Institution of Civil Engineers in Ireland	35 Dawson St., Dublin, Ireland
Bulletin, University of Illinois	University of Illinois Engineering Experiment Station, 106 Engineering Hall, Urbana, Ill.
Bulletin Technique de la Suisse Romande	Case Postale Riponne, 21 Lausanne, Switzerland
Cahiers du Centre Scientifique et Technique du Batiment	33 Rue Jean-Goujon, Paris 8, France
Cement	Sarphatistraat 54, Amsterdam C, Netherlands
Cement and Lime Manufacture	14 Dartmouth St., London (Westminster S.W.1), England

Cement och Betong	Malmo, Sweden
Cemento, Il	9 Via Settembrini, Milan, Italy
Cemento Portland	Calle San Martin 1137, Buenos Aires, Argentina
Civil Engineer Corps Bulletin	Bureau of Yards and Docks, U.S.N., Washington 25, D. C.
Civil Engineering	33 W. 39th St., New York 18, N. Y.
Civil Engineering and Public Works Review	154 Fleet St., London E.C. 4, England
Commonwealth Engineer, The	349 Collins St., Melbourne, C. 1, Australia
Concrete	400 W. Madison St., Chicago 6, Ill.
Concrete and Constructional Engineering	14 Dartmouth St., Westminster S.W.1, England
Constructional Review	14 Spring St., Sidney, N.S.W., Australia
Crushed Stone Journal, The	1415 Elliot Place, N.W., Washington 7, D. C.
Current Road Problems	Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.
Deutscher Beton-Verein	Wisbaden, Germany
Digest	Building Research Station, Garston, Watford Herts, England
Engineering Experiment Station News	Ohio State University, Columbus, Ohio
Engineering News-Record	330 W. 42nd St., New York 18, N. Y.
Giornale del Genio Civile	Via Nomentana 2, Rome, Italy
Hormigon Elastico	Lavalle 1171, 5°, Buenos Aires, Argentina
Handlingar (Proceedings)	Ingeniors Vetenskaps, Akademien, Stockholm, Sweden.
Hungarian Technical Abstracts	Sztálin-Tér 4, Budapest 2, Hungary
Ingenioren	Copenhagen, Denmark
Journal of the American Ceramic Society	2525 N. High St., Columbus 2, Ohio
Journal of the Institution of Engineers Australia	Gloucester & Essex Sts., Sydney, NSW, Australia
Journal of Research	National Bureau of Standards, Washington 25, D. C.
Magazine of Concrete Research	52 Grosvenor Gardens, London S.W.1, England
Osterreichische Bauzeitschrift	Mölkerbastei 5, Vienna 1, Austria
Polytechnisch Tydschrift	28, Balistraat, The Hague, Netherlands
Proceedings, ASCE	33 W. 39th St., New York 18, N. Y.
Public Roads	Bureau of Public Roads, U. S. Dept. of Commerce, Washington 25, D. C.
Reinforced Concrete Review, The	94-98 Petty France, London S.W.1, England
Revista de Obras Sanitarias de la Nation	Charcas 1840, Buenos Aires, Argentina
Revue des Materiaux	19 Rue Lafayette, Paris 9, France
Rock Products	309 W. Jackson Blvd., Chicago 6, Ill.
Structural Engineer, The	2 Bream Bldgs., London E.C.4, England
Technique Moderne-Construction, La	92 Rue Bonaparte, Paris 6, France
Teknisk Tidskrift	Box 84f, Stockholm 2, Sweden
Teknisk Ukeblad	Oslo, Norway
Travaux	6 Ave. Pierre 1st de Serbie, Paris 16, France
Zement-Kalk-Gips	Frankfurterstrasse 5, Wiesbaden, Germany